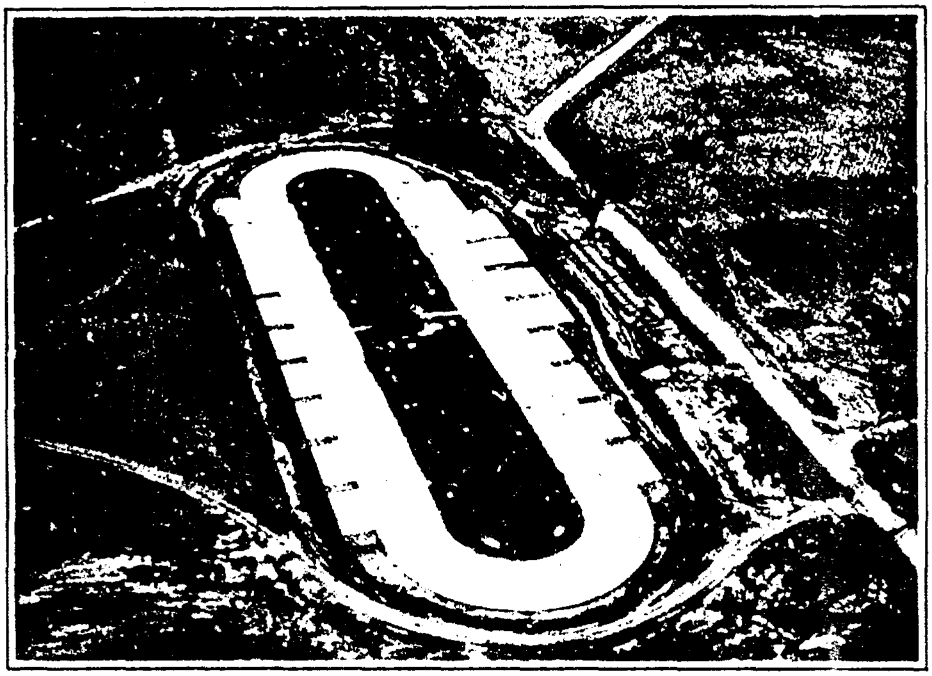
LOCKBOURNE NO. 1
TEST TRACK

FINAL REPORT



WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY

OHIO RIVER DIVISION LABORATORIES

MARIEMONT, OHIO

MARCH 1946

Army Service Forces

Corps of Engineers

LOCKBOURNE NO. 1 - TEST TRACK
FINAL REPORT, ACCELERATED TRAFFIC
TESTS OF CONCRETE PAVEMENTS

Ohio River Division Laboratories
Mariemont, Ohio

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SYNOPSIS

This investigation of rigid pavements was conducted on a specially constructed test track at the Lockbourne Army Air Base near Columbus, Ohio, and forms a part of a series of such investigations being carried out by the Rigid Pavement Laboratory, Ohio River Division Laboratories, Mariemont, Ohio, for the Office of the Chief of Engineers.

The purpose of this investigation was to study the behavior of full scale 6, 8, and 10-inch concrete pavement slabs on different subgrades and bases under 20, 37, and 60 thousand pound wheel load accelerated traffic, as well as a limited number of overlay and steel reinforced concrete slab designs under the same loadings. The principal objectives which have been answered within the limitation of the tests were: to check the method of design outlined in the Engineering Manual, March 1943 edition, to study the effects of types of subgrade, type and thickness of granular base course, type and spacing of joints, and the use of steel reinforcement in concrete pavements, and to study the behavior of concrete overlays under traffic loading.

The pavement of Lockbourne No. 1 was subjected to traffic of earth-moving equipment loaded with concrete blocks to give wheel loads of 20,000, 37,000 and 50,000 pounds respectively. Traffic was continued until test slabs had failed by cracking.

Prior to and during traffic, deflection measurements at the interior, edge, and corner of the test slabs were made under moving and stationary wheel loads by means of the Woodman type electrical gage and mechanical extensometers. Also, at intervals during traffic, cracks and other types of pavement failures were recorded by platting and photographs. A continuous temperature record during traffic was obtained in the top, middle, and bottom of selected test slabs by thermohms installed during construction.

Basic conclusions determined from these studies are as follows:

a. The tentative design curves of the Engineering Manual (March 1943 edition) while nearly correct for the 20,000 pound wheel load, were inadequate for heavier loads.

b. Stresses at the corners and edges of a concrete pavement slab should control the design.

c. Deflections were less for the slabs on the good subgrades.

d. Failure of the slabs occurred on both good and poor subgrades but they occurred much earlier and more rapidly for the slabs on poor subgrade.

e. Even though failure occurred in the slabs on good subgrade, service behavior after cracking was better, in most cases, than that for slabs of greater thickness on poor subgrade.

f. Six inch bases of crushed stone or compacted sand and gravel gave better results than the same thickness of uncompacted sand and gravel.

g. Twelve inch compacted sand and gravel base was superior to a 6-inch compacted base of the same materials.

h. The test indicated that when a concrete pavement is greatly overloaded, a base less than 12-inches thick is not beneficial.

i. Doweling or load transfer devices should be provided for all transverse expansion joints.

j. It was indicated that pavement slabs as small as 10 x 10 feet, and 10-inches or less in thickness are undesirable for pavements designed for wheel loads of 37,000 pounds or greater.

k. The use of steel reinforcement proved beneficial in delaying failure and in prolonging the useful life of overloaded and broken slabs.

l. When overlay slabs having broken base slabs were overloaded, the original crack pattern of the base slab influenced the development of the crack pattern in the top slab.

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SECTION I - INTRODUCTION

1.01 Authority:

The preliminary concepts and approval for the investigation reported herein are contained in two letters from the Office of the Chief of Engineers to the Division Engineer, Ohio River Division, both subject: "Required Thickness of Concrete Pavements". The first letter was of 4 June 1942, and included a first indorsement dated 16 July 1942. The second letter was dated 20 August 1942, and approval of the program outlined in the correspondence is given in the sixth indorsement of 24 June 1943. Final authorization and allocation of funds was given by Directive Consecutive No. A-14793, of 25 June 1943 from the Office of the Chief of Engineers to the Division Engineer, Ohio River Division, subject: "Directive for Tests on Concrete Pavement".

1.02 Purpose and Scope of Investigation:

During the time that the construction and traffic tests were in progress, five reports were prepared covering the construction and the several phases of the traffic testing. Due to this procedure of interim reporting, it was not possible to present a unified report in which data, objectives, and conclusions were organized for continuity and ready reference for the reader. Therefore, it is the purpose of this report to present essentials of construction details, physical data, testing procedures, and test results in a manner which will allow the reader to verify and accurately qualify for his particular use, the conclusions and recommendations given in the synopsis and final paragraphs of this report.

This investigation was instituted to determine the behavior of full scale 6, 8, and 10-inch concrete pavement slabs on various subgrades and bases under 20, 37, and 60 thousand pound wheel load accelerated traffic, as well as a limited number of overlay and steel reinforced concrete slab designs under the same loading.

1.03 Background and Description of the Test Program:

a. Background: The investigation or test program reported herein is preceded by the following three studies conducted in the Ohio River Division, which were reported in *(1), (4), (5), (6), (7), (8), (9), (10), (11), (12), (13), and (14), all reports by this laboratory.

The information contained in the above reports is summarized and discussed in *(1).

These investigations aided greatly in the knowledge gained in the design of rigid pavements; but they also revealed the need for additional pertinent data. This led to the current program of testing, involving the construction of full scale concrete pavement test sections, for the purpose of obtaining the following additional information:

(1) Evaluation of the Method of Design Outlined in Part IV Chapter XX March 1943 edition of the Engineering Manual, for Rigid Pavements.

(2) Evaluation of the Effect of the Following on Pavement Behavior under Repetition of Traffic Loading:

(a) Variation in types of subgrade.
(b) Variation in type and thickness of granular base course.
(c) Variation in type and spacing of joints.
(d) Steel reinforcement in concrete pavements.

(3) Study of the Behavior of Concrete Overlays With Broken and Unbroken Base Slabs Under Repeated Traffic Loading: A further background for this investigation may be obtained from the paper by Robert R. Philippe *(3).

b. Description of Test Program: Investigation of the effect of accelerated traffic tests on full scale concrete pavements was conducted on a specially constructed test track at the Lockbourne Army Air Base, located approximately 15 miles south of Columbus, Ohio. Location, layout, and plan of this test track are shown on Figures 1.0 and 1.1. General features of the plan to be noted were: two continuous traffic lanes 20 feet wide, adjacent 20 x 20 foot slabs of design similar to those in traffic lanes for static load tests, and special subgrades of high bearing value for sections R, S, T, and U, to evaluate subgrade quality. Subgrade for all other sections was natural ground. Concrete units in each tangent section were uniform in design and in thickness, providing combinations of 6, 8, and 10-inch thick pavements on subgrade and base courses 6 and 12-inches thick. This range in thickness of concrete and base treatment was originally considered adequate to bracket design requirements for 37,000 and 60,000 pound wheel loads. On this basis, it had been planned to test the inside traffic lane (Lane 1) with a 37,000 pound wheel load, and the outside traffic lane (Lane 2) under a 60,000 pound wheel load. However, early breakup in Lane 1 under the 37,000 pound wheel load necessitated a lesser wheel load to evaluate the lighter designs. Therefore, Lane 2 for sections A to K inclusive was tested under a 20,000 pound wheel load. This traffic produced well developed cracking in the 6-inch pavements. After formation of these cracks an overlay slab was constructed by placing a 7-inch concrete slab over the cracked pavement of sections A to F inclusive, to carry the

* Numbers in parenthesis refer to bibliography.

60,000 pound wheel load traffic in this lane. The lighter designs, which had concurrently failed in Lane 1 under the 37,000 pound wheel load traffic, were completely replaced with heavier designs to carry additional 37,000 pound wheel load traffic in Lane 1. Details and extent of this reconstruction are shown by Figures 1.5 and 1.6. During the traffic tests, deflection measurements were made under both static and moving wheel loads, and a complete record of crack development was obtained. Traffic was uniformly dispersed over each lane. Figures 2.0, 2.1 and 2.2 are photographs of the equipment used for the 20,000, 37,000 and 60,000 pound wheel load traffic, and indicate schematically the distribution of traffic used in each instance.

1.04 Acknowledgments:

This investigation was carried out under the authorization and direction of the Office of the Chief of Engineers by the Office of the Division Engineer, Ohio River Division. Execution of design details, testing and reporting was the work of the staff of the Ohio River Division Laboratories. Design details were worked out by the Ohio River Division Laboratories personnel and forwarded to the Division Engineer, Ohio River Division, and Office, Chief of Engineers for review. Final designs were those approved by the Office, Chief of Engineers and the Division Engineer, Ohio River Division. The Cincinnati District Engineer Office prepared the specifications and contract drawings, and let the contract for the construction of the Test Track to the V. N. Holderman Construction Company, Columbus, Ohio. Engineering and inspection during construction was performed by personnel of the Ohio River Division Laboratories.

SECTION II - DESIGN AND CONSTRUCTION

2.01 Definitions:

The following definitions specifically apply to the design and construction details of the Test Track.

a. Test Slab: These are 40 x 20 foot units in the tangents of Lanes 1 and 2, bounded transversely at each end by a free or doweled expansion joint, and longitudinally by a free edge and a construction joint which was usually keyed. Test slabs are designated by a system of letters and numbers which indicate their design and location in the Test Track. For example, referring to Figure 1.1, two designations of slabs are given for the two traffic lanes of Section A; they are A1.60 and A2.60. The first letter in each designation refers to the section, the number after the letter to the lane number, the number after the decimal point gives the thickness in inches of the concrete in the slab, while the last number indicates the thickness in inches of the base course; thus, the two slabs are in Section A, Lanes 1 and 2 and are 6-inch thick concrete on subgrade. Further examples are: Slab Q2.1012 is in Section Q, Lane 2, and is a 10-inch thick concrete slab on a 12-inch base course. Slab G2.8R-0 is in Section G, Lane 2 and is an 8-inch reinforced con-

crete slab on subgrade (the "R" indicates general reinforcing). Slab RI.7-60 is in Section M. Lane I, and is a 7-inch concrete overlay on a 6-inch concrete base slab, on natural subgrade.

b. Transition Slab: These are 10 x 20 foot slabs occurring in each of the traffic lanes, and form the transition in design (concrete thickness and/or base course) between the test slabs, as shown on Figure I.I. The slabs were given no specific designation but were referred to as being between specified test slabs.

c. Reconstructed Slab: This refers to the overlays constructed in Sections A through F of Lane 2 after the weaker slabs had failed under the 20,000 pound wheel load traffic, and to the plain and reinforced 8 and 10-inch slabs which replaced the slabs in Lane I after failure under the first part of the 37,000 pound wheel load traffic. Details and extent of this reconstruction are indicated on Figures I.5 and I.6.

d. Overlay Slab: A slab whose vertical thickness is made up of a concrete base slab and a concrete top slab of greater or lesser thickness than the base slab. The base and top slab are either separated by a sand asphalt leveling course or the leveling course is omitted.

e. Reinforced Concrete Slab: A slab which contains either bar mat or wire mesh reinforcement.

f. Subgrade: The natural soil or backfill materials on which a concrete slab or base is constructed. There are three types of subgrade involved in these tests, the natural ground excavated to grade, backfills of compacted granular materials 5 to 6 feet deep, and the 23 inches of replaced compacted subgrade for the reconstructed slabs in Lane I.

g. Base: The term "base" applies to a 5 or 12-inch course of compacted granular materials. Three types of base were used in the test sections, a well graded sand and gravel base, a flume sand base, and a crushed stone base.

h. Free Expansion Joint: These joints were either transverse or longitudinal pavement joints employing a 3/4-inch redwood or premoulded bituminous fiber filler. In all cases a 3/4-inch recess was provided at the top, which was filled with a bituminous joint seal.

i. Doweled Expansion Joint: In all cases, doweled expansion joints were used as transverse joints. These joints consisted of a 3/4-inch premoulded bituminous fiber filler and dowel bars in place across the vertical center of the joint. One end of these dowels was greased and capped to allow movement. Dowels were I-inch in diameter, 16 inches long and placed on 12-inch centers. There were only two exceptions to this use of dowels in expansion joints. These occurred in

the reconstructed slabs where dowel bars 1-1/2-inches in diameter, 16-inches long were placed on 8-inch centers at the jointure of slabs E1.8R-6 and F1.8R-6 to their common transition. In all cases, the filler is the same and a 3/4-inch recess was used at the surface for the joint seal. Sealing of joints was accomplished shortly after completion of the test slab (approximately 5 November 1943).

j. Dummy Contraction Joint: This type of joint was used as a longitudinal and transverse center joint in most of the test slabs. It was formed with a 1/8-inch thick premoulded bituminous ribbon of widths equal to 1/4 of the slab thickness, which was inserted vertically for its full width at the surface of the slab. This ribbon formed the weakened vertical plane for this type of joint.

k. Keyed Construction Joint: In all cases the keyed construction joint was used as a longitudinal joint between Lanes 1 and 2. Details of this joint are shown on Figure 1.3.

l. Doweled Construction Joint: A longitudinal joint used between Lanes 1 and 2, Sections G, H, and J. Smooth 1-inch diameter dowels 16-inches long on 18-inch centers were used, with one end painted and greased.

m. Tangents: These were the straight portions of Lanes 1 and 2. Definitions and test data presented in this report are limited to the pavements of the tangents.

2.02 Design of Test Track:

Pertinent design features of the Test Track are exhibited by Figures 2 to 6 inclusive. These figures show the plan and sections for all test slabs of both the original track and the reconstructed slabs. Shown also are surface and subsurface drainage provisions. The surface of the track is sloped 0.5% toward the center. In the following paragraphs, pertinent design features are described with reference to general objectives of this investigation.

a. Evaluation of Design Methods for Rigid Pavements Outlined in Part IV Chapter XX of the Engineering Manual, March 1943 edition, for Rigid Pavements: The slab designs available in the Test Track for this purpose were the 6, 8, and 10-inch plain concrete slabs on comparable subgrades and/or bases. These slabs were in duplicate sets, such that their performance and design factors could be compared for two wheel loads, either the 37,000 and 60,000 pound wheel loads or the 37,000 and 20,000 pound wheel loads.

b. Evaluation of the Effect of Variations in Types of Subgrade under Repetition of Traffic Loading: There were two types of subgrade which had comparable test slab designs of plain concrete with and without bases. The first type was the natural subgrade which had a series of 6, 8, and 10-inch slabs of plain concrete; and the second type of subgrade was that of sections R, S, T, and U which supported a series of 6-inch plain concrete slabs. There was some variation in the character

and quality of this second type of subgrade, as the subgrade for the four test slabs in sections R and S consisted of about 5 feet of soil stabilized sand and gravel, that of the two test slabs in Section T of about 6 feet of compacted sand and gravel, while that for the two test slabs of section U consisted of 6 feet of compacted sand. However, in all cases these deeply compacted granular subgrades were superior to the natural subgrade. There was also a third type of subgrade used in the reconstructed slabs of Lane I for Sections A through F; but the inclusion of this compacted replaced subgrade was a construction expedient and cannot be evaluated with reference to the original two types, as the reconstructed test slabs on this latter type of subgrade were not comparable to the other designs.

c. Evaluation of Variation in Types and Thicknesses of Base Course Under Repetition of Traffic Loading:

(I) Types of Bases: There were three types of base course used under the series of 6-inch plain concrete slabs in Lanes I and 2 of Sections B through E. The two 6-inch slabs of Section A in Lanes I and 2 were on natural subgrade. All bases were 6-inches thick on natural subgrade. The following table summarizes the variation in types of base course:

Table 2.I

Variation in Types of Base

Slab Designation	Type of Base Course
AI.60	No base, slab directly on natural subgrade
A2.60	No base, slab directly on natural subgrade
BI.66L	Loose or uncompacted sand and gravel
B2.66L	Loose or uncompacted sand and gravel
CI.66S	Compacted flume sand
C2.66S	Compacted flume sand
DI.66	Compacted sand and gravel
D2.66	Compacted sand and gravel
EI.66M	Compacted crushed stone (Macadam)
E2.66M	Compacted crushed stone (Macadam)

(2) Thickness of Bases: Variation in thickness of base was confined to one type (compacted sand and gravel) and two thicknesses of this base (6 and 12 inches) were used in the traffic lanes with the 6, 8, and 10-inch test slabs. These variations are summarized in the following tabulation:

Table 2.2

Variation in Base Thickness

Slab Designation	Thickness in Inches		Type of Subgrade
	Concrete	Base	
A1.60 & A2.60	6*	0	Natural
R1.612 & R2.612	6	12	Compacted Granular
S1.66 & S2.66	6	6	Compacted Granular
F1.80 & F2.80	8	0	Natural
N1.86 & N2.86	8	6	Natural
P1.812 & P2.812	8	12	Natural
K1.100 & K2.100	10	0	Natural
O1.106 & O2.106	10	6	Natural
Q1.1012 & Q2.1012	10	12	Natural

d. Variation in Type and Spacing of Joints: Variation and spacing for the jointing of the tangents of the Test Track were as follows:

Each section was made up of two 20' x 40' slabs separated longitudinally by a keyed construction joint. Transversely the sections were separated at one end by a free expansion joint, and at the other end by a doweled expansion joint. The sections were separated from each other by the 10-foot wide transition slabs which joined the test slabs either with two free or two doweled transverse expansion joints. The test slabs were divided into four 10' x 20' units by longitudinal and transverse dummy joints; and the transition slabs into 10' x 10' units by longitudinal dummy joints. This was the jointing arrangement for sections A through F and N through U as shown on Figure 1.1. Exceptions to this arrangement were present in sections G through K, and the overlay slabs of sections L and M. Further exceptions occurred in the reconstructed slabs as shown on Figures 1.5 and 1.6. This jointing arrangement was planned to make as many test slabs as possible directly comparable insofar as jointing was concerned. The predominate comparison to be observed for the joints was the relative behavior of free and transverse expansion joints under repeated traffic loads.

e. Evaluation of the Effect of Steel Reinforcement in Concrete Pavements: In the original construction there were six 8-inch slabs in sections G, H, and J with steel wire mesh reinforcement in the top of the slabs only. Three different weights of steel were used (68, 91, 159 lbs/100 sq. ft.) (see Figures 1.1 and 1.5). The reconstructed slabs in Lane 1 contained steel reinforcement (see Figure 1.5). The following tabulation summarizes the types and disposition of steel reinforcement in the test slabs:

Table 2.3

Variation in Steel Reinforcement

Slab Designation	Type of Reinforcement	Spacing, Inches		Weight in lbs./sq. ft.	
		Top	Bottom	Top	Bottom
	Original Slabs				
G1.8R-0	Wire Mesh (1)	6 x 6	None	68	None
G2.8R-0	" "	6 x 6	"	68	"
H1.8R-0	" "	6 x 6	"	91	"
H2.8R-0	" "	6 x 6	"	91	"
J1.8R-0	" "	16 x 2	"	159	"
J2.8R-0	" "	16 x 2	"	159	"
	Reconstructed Slabs				
A1.10S	None	--	--	--	--
B1.10R-6	Wire Mesh (1)	6 x 6	6 x 6	68	68
C1.10R-6	" "	6 x 6	6 x 6	156	156
D1.10R-6	" "	6 x 6	6 x 6	68	68
E1.8R-6	" "	6 x 6	6 x 6	156	156
F1.8R-6	Bar Mat (2)	8 x 8	8 x 8	193	193
F1.8R-6	Bar Mat (2)	6 x 6	None	259	None

(1) 2-inch clearance top and bottom
(2) 1.5-inch clearance; 0.5-inch diameter bars

The original slabs had only light reinforcement in the top in the weights and distribution generally used for concrete pavements.

f. Study of the Behavior of Concrete Overlays on Broken and Unbroken Base Slabs under Repeated Traffic Loading: The overlay designs for this study were included in the original construction in Lanes 1 and 2, Sections L and M, and in Lane 2 Sections A through F for the reconstructed slabs, as shown on Figures 1.1 and 1.5. The following table summarizes the pertinent features of these designs:

Table 2.4

Variation in Overlay Designs

Slab Designation	Thickness in Inches			Condition of Base Slab	No. of Coverages on Base Slab
	Base Slab	Overlay	Leveling Course (1)		
		Original Slabs *			
L1.5-60	6	5	0.75	Free of Cracks	0
L2.5-60	6	5	0.75	" " "	0
M1.7-60	6	7	0.75	" " "	0
M2.7-60	6	7	0.75	" " "	0
		Reconstructed Slabs **			
A2.7-60	6	7	0.75	*** Partially cracked	520
B2.7-66L	6	7	0.75	Badly cracked	524
C2.7-66S	6	7	0.75	Badly cracked	526
D2.7-66	6	7	None	A few cracks	554
E2.7-66M	6	7	None	A few cracks	556
F2.7-80	8	7	None	Free of cracks	554

(1) When a leveling course is indicated, sand asphalt was used.
* Original slabs constructed into overlay slabs without exposure of base slab to traffic.
** Top slab constructed after indicated coverages of traffic on base slab.
*** See Figures 5.0 and 5.1 for graphical representation of crack pattern of base slabs before construction of top slab.

Reconstruction in Lane 2 consisted of placing a 7-inch concrete top slab over the original slabs of sections A through F. The cracks in the original or base slabs of these sections resulted from the 20,000 pound wheel load traffic. The character and extent of the cracks are given by the final crack pattern of Figures 5.0 and 5.1 for the 20,000 pound wheel load traffic. The jointing of the overlays was identical to that of the base pavement with the one exception that the transverse joints of the transition slab between the test slabs D2.7-66 and F2.7-66M lapped those of the base pavement by 18 inches as shown on Figure 1.5.

In general, the overlay slabs were designed to give the following information:

(1) Comparison to single slabs of equivalent design.

(2) Effect of a sand asphalt leveling course between the base slab and top slab.

(3) Effect of cracks in the base slab on the behavior of the top slab.

2.03 Construction:

Construction of the Test Track, as shown on Figure 1.1 was started on 2 August 1943 and completed 3 November 1943. The reconstruction, as shown on Figure 1.5, was done between 5 June and 10 July 1944. Construction methods used for the subgrades, bases, and concrete follow:

a. Subgrades: Two types of subgrade were used in the initial construction of the Test Track, the natural subgrade, and the deep compacted granular subgrades of sections R, S, T, and U. In the reconstructed slabs an additional type of subgrade was used which is designated as a compacted replaced subgrade.

(1) Natural Subgrade: This type of subgrade was always prepared by excavation to about grade by means of scrapers, after which forms were placed and excavation to exact grade was done by hand methods followed by a light rolling with a smooth wheeled roller. Determinations of unit weight and water content, and field bearing tests were made on the finished subgrades before concrete or bases were placed.

(2) Deep Compacted Granular Subgrades: Deep sections were excavated in the natural ground to about 6 feet below grade. These sections were then backfilled to grade with granular materials which were spread and compacted in 6-inch layers. Compaction was accomplished with a sheeps-foot roller having a foot pressure of about 215 $lbs/in.^2$. The materials in these sections were sampled during construction for in-place determinations of unit weight and water content. Also, field bearing tests were made on the finished subgrades before concrete or bases were placed.

(3) Compacted Replaced Subgrade: This type of subgrade was used for the reconstruction of the test slabs in Lane 1 of sections A, B, C, D, E, and F. Construction consisted of first removing the broken concrete, base, and natural subgrade to a depth of about 2 feet below grade. This excavation was then backfilled to grade by spreading and compacting the cohesive soil available at the site in 6-inch layers by means of a bulldozer and a sheeps-foot roller. The pressure on the feet of the roller was about 215 $lbs/in.^2$. The soil was maintained as nearly as possible at its optimum moisture content during compaction.

b. Bases: The three types of bases constructed were sand, crushed stone, and the compacted sand and gravel base. Unit weight and water content determinations were made of the finished bases in place.

(1) Sand Base: This was a 6-inch compacted sand base. The sand was the same as that used in the deep granular subgrade of section U. The rolling and compacting of the sand in the base by the usual methods was not entirely successful, due to the fact that there was only one 6-inch layer over the natural subgrade. A fair degree of compaction was achieved by thoroughly wetting the sand with water. Despite wetting of the sand, stability of the subgrade was not effected.

(2) Crushed Stone Base: This base consisted of a 6-inch compacted layer of crusher-run stone. The crushed stone was placed on the natural subgrade in a 3-inch layer which was wetted and rolled with a smooth two-wheeled 7.5 ton roller. The surface was choked with screenings and again flat rolled with two additional passes of the roller. A second 3-inch layer of crushed stone was placed, wetted, rolled and choked with screenings in the same manner as the first layer.

(3) Sand and Gravel Base: This type of base was 6 and 12-inches thick and was constructed either by spreading and compacting one 6-inch layer of material or two 6-inch layers on the subgrade. One exception to this procedure was made in section B where the 6-inch base was only spread and leveled before placing concrete. This was designated as the loose sand and gravel base. Compaction of the other sand and gravel bases was achieved first by wetting the material, then by utilizing the combined weight of the roller and caterpillar tractor towing it. Reconstructed bases were compacted in the same manner. Unit weight, water content, and base moduli were measured for the completed base courses before concrete placement.

c. Concrete: Transit mixed concrete containing natural sand and gravel (1-1/2 inc maximum size) was used. The concrete mix design was based on 5-1/2 sacks of cement per cubic yard of concrete. Water content varied from approximately 5-1/2 to 6 gallons of water per sack of cement. Each batch of concrete was inspected visually before placing, and in certain instances it was found necessary to add water to obtain the desired workability. This factor accounted for the wide variation in water content previously noted.

Concrete used in the reconstructed slabs differed from the concrete used previously in that High Early Portland Cement was used in place of the Normal Portland Cement and a higher water cement ratio was used.

Curing of the concrete was accomplished with the use of wet cotton mats. Concrete in the reconstructed slabs was cured by using damp straw. (See (2) for further details concerning concrete construction).

Concrete specimens for testing were as follows:

(1) Control Beams and Cylinders: Three to four 4 x 4 x 16-inch beams were cast from representative concrete taken from each test slab. Each day, three 6 x 12-inch cylinders were cast from representative concrete. These were tested in the laboratory as explained later in this report (see (2)).

(2) Field Beams: For each test slab, two field beams having a width and depth equal to the slab thickness and a length equal to four times the slab thickness, were cast in wood forms on representative subgrade or base near each section. These beams were cured by the same method and condition as the corresponding test slab, and were left

in the field until time for testing. They were tested for flexural strength and other properties at about the time the test slabs were tested under traffic loading.

SECTION III - TRAFFIC AND AUXILIARY TESTING

3.01 Definitions:

The following definitions apply specifically to the traffic and auxiliary testing of the Test Track:

a. Deflection: Vertical movement of a slab at any point at its interior, edge, or corner resulting from moving or stationary wheel loads only.

b. Tire Print: Contact area of tire on pavement when wheel is stationary. (See typical tire prints on Figures 2.0, 2.1, and 2.2).

c. Trip: One passage of the loading equipment over a slab.

d. Coverage: The planned routing of two trips of the loading equipment to produce as uniform a distribution of traffic as possible over a slab, as shown on Figures 2.0, 2.1, and 2.2.

3.02 Traffic Loading Equipment:

Two types of heavy earth moving equipment were used, each consisting of a scraper unit and a power unit. The following table indicates the type used for each wheel loading:

Table 3.1

Dimensional Data of Traffic Loading Equipment

Wheel Load In Lbs.	Tire Size In Inches	Tournapull	Scraper	Speed of Travel
20,000	21 x 24	Super C	Model L.P.	5 to 10 m.p.h.
37,000	30 x 40	A - 3	Model N.U.	4 to 5 m.p.h.
60,000	30 x 40	A - 3	Model N.U.	3.0 to 3.5 m.p.h.

3.03 Traffic Coverage of Test Track:

The major portion of the traffic was completed during the summer and fall of 1944. This included the 20,000, 37,000, and 60,000 pound wheel load traffic, which adequately established the degree to which the majority of the test slabs in the Track could carry these wheel loadings. The only exception was the heavily reinforced designs for the reconstructed slabs in Lane I; therefore, to more fully develop the potentialities of these slabs, they were subjected to 1084 addi-

tional coverages of 55,000 pound wheel load traffic during September 1945. The tire print, and distribution per coverage for the 55,000 pound wheel load are essentially the same as that shown for the 60,000 pound wheel load traffic on Figure 2.2.

The following table gives the schedule and extent of traffic coverage for the Track:

Table 3.2

Traffic Schedule and Coverages

Traffic Wheel Load in Lbs.	Location	Time of Traffic		Number of Coverages
		From	To	
20,000	North Tangent Lane 2	16 May '44	27 May '44	537
37,000	Lane 1	18 Apr. '44	9 May '44	363
37,000	Lane 1	19 July '44	6 Sept.'44	1118
37,000	Lane 1	29 Nov. '44	9 Dec. '44	804
				2285 Total
60,000	Lane 2	11 Sept.'44	25 Nov. '44	712
55,000	Lane 1, Sections A through F	17 Sept.'44	18 Oct. '45	1084

3.04 Data Obtained During Traffic Tests:

a. Stationary Wheel Load Deflection Measurements: These measurements were at joint, edge, and interior positions using the appropriate wheel load for each slab involved. They consisted of mounting extensometer gages on a wooden cantilever and running a wheel of the loading equipment alongside the extensometer as shown by the photographs of Figure 2.3. The equipment was stopped in this position and the deflection indicated by the extensometer was read after one minute; the equipment was then moved to the next position. The sequence of these measurements is clearly indicated by the diagramatic sketches accompanying the results which are given on Figures 4.0 through 4.7.

b. Moving Wheel Load Deflection Measurements: These measurements were taken at a few points at interior slab positions and across joints for the 20,000, 37,000 and 60,000 pound wheel loads. Woodman electrical deflection gages were used with an amplification circuit and recording oscillograph to obtain a continuous record of deflection at the gage point as a wheel of the loading equipment approaches, passes over, and leaves the gage. Necessary details of gage location and direction of traffic are shown schematically with the results of these measurements given on Figures 4.8 to 4.11. Difficulties with the amplification circuits, and the mechanical functioning of the gage severely limited its use when applied to measuring deflections of concrete pave-

ment slabs. These gages were designed and built by the United States Waterways Experiment Station, Vicksburg, Mississippi for measuring deflections of flexible pavements, and were considered the best available for measuring deflections of rigid pavements under moving wheel loads at the time of the tests. Details of the gage, method of installation, and circuits are given on Figures 2.4, 2.5, and 2.6.

c. Other Measurements: In addition to the stationary and moving wheel load deflection measurements, the crack patterns were mapped and photographed as they developed in the test slabs and transition slabs under the repetition of traffic loading. Records of daily temperatures were obtained from the weather station at the base. Thermohm installations in the top, middle, and bottom of 6, 8, and 10-inch slabs were used with a 6 point micromax recorder to obtain continuous readings of temperature change within the concrete.

SECTION IV - PHYSICAL PROPERTIES

4.01 General:

This section of the report summarizes the pertinent physical properties of the subgrades, bases, and concrete as they were at the time of construction, and at the time of traffic testing. The physical properties of the subgrades and bases were obtained from preliminary auger borings covering the area on which the Test Track was constructed and from tests made during construction. These tests were adequate to fix the soil classification, degree of compaction, and water content at the time of construction; but did not necessarily fix the bearing values of the subgrades and bases at the time of traffic testing which was in progress 6 months to 1 year after completion of the original construction. In fixing the bearing value of the subgrades and bases at the time of traffic tests, water content determinations of base and subgrade materials beneath the test slabs were made by sampling these materials through core holes drilled in the slabs. These water content determinations showed conclusively that the water contents of the subgrades at the time of traffic testing were essentially the same as they were at the time the field bearing tests were made during construction. Therefore, it is assumed that the subgrade and base course moduli measured during construction were also representative at the time of the traffic tests. This assumption is further substantiated by one bearing test which was repeated after removing a test slab which failed during traffic, and by static loading tests on the static test slabs. These static loading tests were made during and after the traffic tests. Coring of the test slabs also provided information on the actual thickness of the concrete, which was used in theoretical computations of stress in the test slabs for the appropriate loadings. (See Section V - Computations). It was found that the slab thicknesses were as much as 0.4 to 1.0 inch less than the design thickness for the original construction; while for the reconstructed slabs, actual thickness and design thickness corresponded very closely.

Where bearing values (subgrade or base course moduli "k" in lbs/in.^{3}) are given in this report, they were determined by the field bearing test procedure given in Chapter XX of the March 1943 edition of the Engineering Manual, without correction for saturation. Physical properties of the concrete were determined from control beams and cylinders during construction, and from field beams and beams sawed from broken pieces of the test slabs at the time of the traffic tests.

4.02 Subgrades:

The following table summarizes the average physical properties of the three general types of subgrade present in this investigation:

Table 4.1

Average Physical Properties of Subgrades

Type of Subgrade	Average M.I.T. Classif. % Clay	% Silt	% Sand	% Gravel	Plasticity Index	Water Content % Dry Wt.	Unit Dry Wt. #/ft.^{3}	"k" #/in.^{3}
Natural	30	55	15	--	20	20	105	90
Reconstructed	25	50	25	--	13	15	110	95
Deep Granular								
Sand & Gravel	--	--	40	60	N.P.	7	132	371
Soil Stabilized	7	13	40	40	7.3	9	125	400
Sand	--	--	50	50	N.P.	5	122	207

The sand and gravel and the soil stabilized sand and gravel of the deep granular subgrades had a maximum gravel particle size of 2.5 inches, while that of the sand subgrade (flume sand) was about 5.0 mm. or 0.2 inches.

The natural overlying soil and topography of the site are the result of the Wisconsin stage or glaciation. The glacial till extends to considerable depths. The upper 10 to 15 feet of this material is predominately a grayish brown silty clay, containing traces of sand and gravel with a few small to medium sized boulders.

4.03 Bases:

The average physical properties of the varied types of bases used in this investigation are summarized in the following tabulation:

Table 4.2

Average Physical Properties of Bases

Type of Base	M. I. T. Classif.		Maximum Grain Size	Water Content	Unit Dry Wt. lbs/ft.^{3}	"k" lbs/in.^{3}
	% Sand	% Gravel				
Crushed Stone	22	78	0.8"	5	133	108
Sand	50	50	0.2"	6	117	72
Compacted Sand and Gravel	40	60	2.5"	6	130	90
Reconstructed Sand & Gravel	50	50	2.5"	5	126	192

The 6-inch crushed stone base was compacted to about 95% of its modified A.A.S.H.O. maximum unit dry weight, but no appreciable increase in bearing value was obtained over that of the natural subgrade on which it was placed.

The compacted sand and gravel bases varied in degree of compaction from 90% to 95% of the modified A.A.S.H.O. maximum unit dry weight for these materials. The 6 and 12-inch bases placed on natural subgrade did not increase the subgrade modulus; i.e., the base and subgrade modulus measured were essentially the same. However, the 6-inch sand and gravel bases placed on the reconstructed subgrade showed a considerable increase in measured values of base modulus over those of the subgrade.

4.04 Concrete:

a. Concrete Mix Design: The concrete mixture used in the original construction consisted of regular Portland Cement (Federal Specification SS-C-191b), natural sand and 1-1/2-inch maximum size gravel in the weight proportions of 1:2.34:4.12. The cement content was 5.5 sacks per cubic yard of concrete, and the water-cement ratio ranged between 5-1/2 and 6 gallons of water per sack of cement.

The same concrete mixture was used for the reconstructed slabs except that the cement was high-early-strength Portland (Federal Specification SS-C-201a) and the water-cement ratio ranged between 6 and 6-1/2 gallons of water per sack of cement. This higher water content was necessary because of the use of high-early-strength cement during a period of unusually high air temperatures, and because of the slow progress in placing the concrete in the slabs reinforced with two layers of steel.

b. Test Specimens: Concrete test specimens consisted of laboratory-cured concrete control beams and cylinders, field-cured beams, and sawed beams.

(1) Control Specimens: For the original construction the control specimens consisted of one set of three or four, 4 x 4 x 16-inch beams for each 20 x 40-ft. test slab, while for the reconstructed slabs each test slab was represented by two, 4 x 4 x 16 inch beams. In addition one set of two or three 6 by 12-inch cylinders was cast each day that concrete was placed. These specimens were cured under damp cotton mats in the field until they were transported to the Ohio River Division Laboratories, generally within a week after they were cast. Curing was then continued in the laboratory fog-room.

(2) Field Beams: During both the original construction and the reconstruction, two concrete beams, having widths and depths equal to the thickness of the pavement represented and lengths of four times the thickness, were cast on representative subgrade or base for each test slab. These specimens were cured in the same manner as the pavement. However, the field beams for the original construction were not protected against freezing during the winter of 1943-44 when the pavement slabs were covered with straw and tarpaulins.

(3) Sawed Beams: Beams having widths and depths equal to the thickness of the slab and lengths of four times the thickness were sawed from pieces of a number of slabs which had failed during the traffic tests.

c. Tests: The following tests were conducted on specimens representing the concrete in the test slabs:

(1) Slump
(2) Dynamic Modulus of elasticity
(3) Static modulus of elasticity
(4) Flexural strength
(5) Compressive strength
  (a) Beam specimens tested as modified cubes
  (b) Cylinder specimens
(6) Density
(7) Absorption

Whenever applicable, the standard procedures of the American Society for Testing Materials were used in conducting the tests.

The dynamic modulus of elasticity of the beam specimens was determined by the electrical method of flexural vibration. A detailed description of the dynamic method employed for this test appears in an article by Obert and Duvall (20).

d. Test Results: A summary of the results of tests conducted on the specimens representing the concrete in the test slabs subjected to traffic tests is presented in the following table:

Table 4.3

Summary of Test Results

Type of Specimen Tested	Slump Inches	Number of Specimens.*	Avg. Age Days	Modulus of Elasticity psi. x 10^6 Dynamic	Modulus of Elasticity psi. x 10^6 Static	Strength, psi. Flexural	Strength, psi. Compressive
20,000 Pound Wheel Load Traffic, North Tangent - Lane 2							
4 x 4 x 16 inch Beams	2½	33	28	5.50	3.36 (1)	660 (28)	4130 (23)
6 x 12 inch Cylinders		2	28	--	3.75	--	5005
Field Beams		20	300	5.34	3.53 (19)	730 (16)	5710 (18)
Sawed Beams		3	540	6.05	3.87	880	--
37,000 Pound Wheel Load Traffic - Lane 1							
a. Original Construction							
4 x 4 x 16 inch Beams	3	74	28	5.16	3.40 (4)	650 (64)	4210 (59)
6 x 12 inch Cylinders		6	28	--	3.44	--	4085
Field Beams		30	300	5.59	3.77	780	6000
Sawed Beams		18	475	5.60	3.68	790	5475 (12)
b. Reconstructed Slabs							
4 x 4 x 16 inch Beams	4	12	14	4.64	--	660	4900
		2	7	4.07	--	555	4205
6 x 12 inch Cylinders		5	14	--	--	--	4195
Field Beams		14	51	5.08	3.27	750	4920 (12)
60,000 Pound Wheel Load Traffic - Lane 2							
a. Original Construction							
4 x 4 x 16 inch Beams	2½	71	28	5.44	3.40 (5)	625 (64)	4310 (54)
6 x 12 inch Cylinders		8	28	--	3.73	--	4390
Field Beams		44	330	5.50	3.66 (43)	720 (38)	5900 (38)
Sawed Beams		16	570	6.12	3.82	905	6125
b. Reconstructed Slabs							
4 x 4 x 16 inch Beams	5	6	14	4.40	--	665	5360
		4	7	4.35	--	585	4445
6 x 12 inch Cylinders		2	14	--	--	--	4430
		2	7	--	--	--	4070
Field Beams		12	136	5.17	--	760	5070

*Number of specimens represented except as otherwise indicated by numbers in parenthesis.

Density and absorption tests which were conducted on pieces of broken beam specimens showed the following results:

Average density 152.4 pounds per cubic foot.
Average absorption 6.1 percent by weight of the concrete.

The following is a tabulation of the average flexural strength values together with the respective estimated 28 day strength values summarized from the preceding table:

Table 4.4

Average Flexural Strength Data of Concrete

Location	Laboratory Beams 4x4x16 in.			Field Beams			Sawed Beams		
	Age Days	Avg. psi.	Avg. Estimated 28-day Strength psi. (1)	Avg. Age Days	Avg. psi.	Avg. Estimated 28-day Strength psi.	Avg. Age Days	Avg. psi.	Avg. Estimated 28-day Strength psi.
			Original Construction						
20,000# Wheel Load	28	660	615	300	730	595	540	880	695
37,000# Wheel Load	28	650	605	300	780	635	475	790	630
60,000# Wheel Load	28	625	580	330	720	585	570	905	710
			Reconstructed Slabs						
37,000# Wheel Load	14	660	615	51	750	(2)	--	--	--
60,000# Wheel Load	14	665	620	136	760	(2)	--	--	--

(1) For standard beams of 6-inch square section with 3rd point loading.

(2) No data available for estimating 28-day strength of concrete containing high-early-strength cement.

e. Discussion: With respect to the physical properties of the concrete, the results of flexural strength tests and of modulus of elasticity determinations are significant in evaluating the results of the traffic tests.

Average values of flexural strength of the different beam specimens are tabulated in Tables No. 4.3 and No. 4.4. The summary in Table No. 4.4 includes estimated 28-day strength values for specimens tested at later ages. Also, the strengths obtained with 4 x 4 x 16-inch beams are corrected to be comparable to values obtained with standard beams having a 6-inch square section. The size of the field beams and sawed beams varied with the thickness of the pavement represented; however, no strength corrections were made for the different sized beams.

All traffic test slabs for the original construction, except slabs G, H, J, and K and the overlay slabs in section L and M, were cast during the first two weeks after concrete construction was commenced. The average flexural strength of the concrete control beams cast during this two-week period was 620 psi, with 90% of the values within 10% of the average for all sets cast. The average flexural strength of the control specimens from the remaining test slabs was approximately 10% higher, with similar uniformity of the concrete.

For the reconstructed slabs the control beams had an average flexural strength of 660 psi. at 14 days, and the results were slightly less uniform than for the original construction. This may be attributed partly to the difficulties encountered during construction, and to the fact that the specimens were field-cured for 7 days or longer before being sent to the laboratory. In addition, high-early-strength cement was used, during a particularly dry spell, and difficulty was encountered in maintaining the proper workability during concrete placement.

The specimens show continuous gain in flexural strength with age, and the sawed beams indicate that flexural strength of the concrete in the test slabs was at least as great as that of the cast specimens representing this concrete. It is for note that estimated 28-day strengths of control beams (estimated on basis of cross section) and field beams (estimated on basis of age) are comparable. On the other hand, estimated 28-day strength (estimated on basis of age) of sawed beams, was found to be higher.

For the original construction the average modulus of elasticity of the concrete beam specimens at the age of 28 days was approximately 5.3 million psi. in the dynamic tests, and 3.4 million psi. in the static tests. At the time of the traffic tests the modulus increased to approximately 5.6 million psi. in the dynamic tests and 3.7 million psi. in the static tests.

SECTION V - COMPUTATIONS

5.01 General:

For purposes of evaluation, comparisons, and estimates of the degree of overload, three computations of the maximum theoretical stress have been made of each slab for the wheel load or wheel loads involved.

Two of these computations are for the wheel loading at the interior portion of the slab (i.e., at considerable distance from an edge or joint) and the third computation is for the maximum stress at a loaded transverse edge.

5.02 Interior Loading - 1st Method:

The first computation for maximum stress at the interior position is based on the formulas of paragraphs 20-50 c. Part IV, Chapter XX of the Engineering Manual, March 1943 Edition. The tentative design curves given by Exhibit I, sheets I, 2, and 3 of Part IV, Chapter XX of the March 1943 Edition of the Engineering Manual are based on these formulas. The original sources of these formulas is a paper by H. M. Westergaard (15).

5.03 Interior Loading - 2nd Method:

The second method of computing maximum stress for loads at the interior portion of the slabs is based on the mathematical analysis for a large slab on an elastic subgrade offering a reaction per unit of area equal to the deflection times a constant "Modulus of Subgrade Reaction" k, the place of the load being far from any edge. This analysis is given by H. M. Westergaard (17).

5.04 Edge Loading:

The third computation is for the maximum stress in a panel of concrete pavement, with a straight edge, loaded near the edge through the footprint of a tire, supported at all points by a conforming subgrade or base, but with no support of the edge by the adjacent panel. The analytical formulas for treating this case of edge loading are taken from a Progress Report of Commander H. M. Westergaard, CEC-V (S), USNR. (18). On 24 June 1944 a copy of this report was sent to the Office of the Chief of Engineers by the Chief of the Bureau of Yards and Docks of the Navy Department. As this report is unpublished at present, the pertinent formulas will be listed herein with definition of terms for reference: The formulas for the maximum stress produced by edge loading are:

$$\sigma e = \frac{12(1+\mu)P}{\pi(3+\mu)h^2}\left[K + 0.8559 - \frac{\mu}{4} - B_1 - \frac{(1-\mu)}{4}\,S + \frac{B_2\,\bar{y}}{\ell}\right] \quad -- \quad 1$$

wherein

σe is the maximum tensile stress at the bottom of the slab.
P = load
h = thickness of pavement
μ = Poisson's ratio for the concrete
$\bar{y}$ = distance from the edge to the center of gravity of the load
ℓ = radius of relative stiffness defined by equations 2 and 44 in Westergaard (17).

K and S = area coefficients defined by equations 11 and 12 in (17) for the point x = y = 0, with the axis of x along the edge, and with c = ℓ; the computation of K and S is shown in a number of cases in Part IV Chapter XX of the March 1943 edition of the Engineering Manual.

B_1 and B_2 = constants defined by the following formulas 2 and 3

$$B_1 = 2\int_0^\infty \frac{\left[1 + (1-\mu)^2 \alpha^2 \beta^2\right] \beta \, d\alpha}{1 + 4(1-\mu)\alpha^2 \gamma^2 - (1-\mu)^2 \alpha^4} \qquad 2$$

$$B_2 = 1/2 \int_0^\infty \frac{\left[1 + \mu - 2(1-\mu)^2 \alpha^2 \beta^2\right] d\alpha}{1 + 4(1-\mu)\alpha^2 \gamma^2 - (1-\mu)^2 \alpha^4} \qquad 3$$

in which

α, β, and γ are ratios obeying the relations:

$$\alpha^2 + \beta^2 = \gamma^2 \text{ and } \beta\gamma = 1/2 \qquad 4$$

5.05 <u>Overlay Slabs, Reference (19):</u>

The maximum stresses in the base of the overlay slabs were computed by the single slab formulas of (15), (17), and (18). This was done by obtaining an equivalent single slab thickness from the following empirical relationship:

$$h_1 = \sqrt{h^2 - C h_2^2} \qquad 5$$

in which

- h = the equivalent single slab thickness
- h_1 = thickness of the overlay slab
- h_2 = thickness of the base slab
- C = a coefficient whose value is dependent on the condition of the base slab as follows:
  - C = 1.00 if the base slab is in good condition
  - C = 0.75 if base slab contains initial joint and corner cracks due to loading but no progressive cracks
  - C = 0.35 if base slab is badly cracked and crushed

The above relationship is taken from (19).

5.06 <u>Application and Limitations:</u>

The application of the above methods of computing maximum tensile stresses in the base of concrete pavement slabs are all subject to the limiting assumption of elasticity for the concrete slab and supporting medium. In addition, the supporting media, for the cases under consideration, is either subgrade or 6 and 12-inch base courses; and this assumption, as used, further implies this media furnished a constant "modulus of reaction" (reaction per unit of area of deflection).

The physical properties of the concrete, bases, and subgrade, required by the above methods of maximum stress computation, are the subgrade or base modulus of reaction "k", the modulus of elasticity "E" of the concrete, and Poisson's ratio "μ" of the concrete. The modulus "k" differs for each slab and is given for each slab with the results of the stress computation. However, an overall value of 4×10^6 lbs/in.^{2} is taken for "E" and similarly a value of 0.20 for "μ". Equations 2 and 3 of Reference (18), which are functions of "μ", are evaluated for "μ" = 0.20. The values of B_1 and B_2 obtained for μ = 0.20 are:

$$B_1 = 0.9627$$
$$B_2 = 0.4145$$

In addition to these physical properties, the dimensional properties, as load, size of tire print, and thickness of the pavement are necessary. The values used are essentially those of the test conditions, and are as shown in the following table:

Table 5.1

Dimensions of Typical Tire Prints

Wheel Load in lbs.	Tire Print		
	Length in inches	Width in inches	Area Sq. inches
20,000	26.26	17.56	362.17
37,000	34.76	23.38	638.29
60,000	42.00	26.00	857.66

For the computations by the formulas of Reference (15), the assumption is made that the load is transmitted to the pavement by an equivalent circular area, while for those of (17) and (18) the loaded area is taken as an ellipse having for its major and minor axes the length and width of the tire print. The actual outline of the tire for each loading is shown on Figures 2.0, 2.1, and 2.2. The thickness "h" used for each slab is a measured thickness at both the interior and edge and is given with the results of the computations.

The computations of maximum stress are finally used as a means of computing "design factors" for the conditions of test of each slab. This was accomplished by dividing the average flexural strength of the concrete in the slab at the time of the traffic testing by the three computed maximum stresses, thus giving three "design factors" for each slab, two on the basis of interior loading and one based on edge loading. The values of flexural strength used were based on the tables and discussion in paragraph 4.04.

SECTION IV - WEATHER CONDITIONS AND CONCRETE TEMPERATURES DURING CONSTRUCTION AND TRAFFIC TESTING

6.01 Sources of Temperature and Precipitation Data:

a. Precipitation and Ambient Air Temperatures: A record of the daily maximum and minimum air temperature and amount of precipitation was obtained from the Lockbourne Army Air Base weather station located about a half mile from the pavement testing site. These data are presented in the charts of Figures 3.0 and 3.1, and the periods of construction and testing are designated on the charts.

b. Temperatures of the Concrete in the Pavement: Electrical resistance thermometers, called thermohms, were installed in the 6, 8, and 10-inch pavements, which were laid on natural subgrade and on base course. The thermohms were installed in groups of three in the interior of the slab; one in the top of the slab was placed 5/8-inch from the pavement surface, one was placed in the mid-plane, and the remaining one was placed 5/8-inch from the bottom surface of the slab. The wires leading from the thermohms were carried into the field office to a six-point "Micromax" recorder. The "Micromax" recorded the resistance change in the thermohms due to temperature changes in the concrete. The resistance values were converted to temperature in degrees by means of calibration curves. The "Micromax" provided a continual record of the temperatures in two slabs simultaneously. The recording of pavement temperatures was started in November, 1943 and extended to July, 1945.

6.02 Temperatures and Precipitation During Construction and Testing:

a. Original Construction: During the three month period of original pavement construction, the precipitation was light and the temperatures mild; factors conducive to good pavement construction. The precipitation during the construction period amounted to 4.9 inches, of which one inch occurred during the second day after the project was started. The average ambient air temperature during the period of concrete placement (30 September-29 October 1943) was 53° F; and all concrete was placed in temperatures above freezing.

b. Reconstructed Slabs: During reconstruction of Lanes 1 and 2, sections A through F (5 June-6 July 1944), the weather was ideal for construction. The average ambient thmperature was 74° F and the precipitation negligible (0.6 inches).

c. Temperature and Precipitation During Test Periods: Between 16 November 1943, when the recording of pavement temperatures was started, and 2 December 1943, when the track was covered with straw and tarpaulins, the temperatures in the top of the pavement frequently reached freezing. About 2 December 1943, just before the track was covered, temperatures in the bottom of the slabs went down to between 30 and 32 degrees Fahrenheit.

Throughout the winter, after the pavement was covered, temperatures in the concrete averaged 35° to 40° F. with infrequent drops to freezing temperatures. During this period, the temperature differential between the top and bottom of the slabs was small with very little daily variation.

The amount of precipitation occurring during the spring of 1944 was about normal. During the summer and fall, precipitation was considerably less than normal until the month of December, when it approached twice the normal for that month.

The following tabulation indicates the maximum temperature differentials that were measured during test periods from April to December 1944 by the thermohm installations in the concrete slabs. No record of pavement temperatures was kept during the period of the 55,000 lb. wheel load traffic from 17 September to 18 October 1945.

Table 6.1

Maximum Temperature Variation in Concrete Slabs

Date 1944	Time Hours	Thickness of Slab Inches	Temperature in °F			Diff. Top and Bottom
			Top	Middle	Bottom	
21 April	1600	6	81	69	64	17
2 May	1600	6	90	80	74	16
9 May	1600	10	80	65	63	17
21 May	1400	10	98	82	75	23
27 May	1600	6	103	89	80	23
27 May	1600	8	103	91	84	19
21 July	0800	8	65	71	75	-10
23 July	1400	8	102	89	84	18
30 July	1600	8	101	89	83	18
13 Sept.	0600	8	62	64	65	- 3
13 Sept.	1600	8	85	76	71	14
18 Sept.	1600	8	91	81	75	16
18 Sept.	1600	6	92	83	79	13
25 Sept.	0800	8	53	58	62	- 9
25 Sept.	0800	6	53	57	61	- 8
16 Oct.	0400	8	45	51	55	-10
16 Oct.	0400	6	44	49	53	- 9
6 Nov.	0400	10	37	41	46	- 9
3 Dec.	0800	8	23	27	32	- 9
3 Dec.	0800	6	23	26	31	- 8
3 Dec.	1500	8	37	36	33	4
3 Dec.	1600	6	35	34	33	1
4 Dec.	0800	8	24	28	31	- 7
4 Dec.	1500	8	40	37	34	6

The above tabulation indicates the following with respect to pavement temperature differentials measured during test periods:

(1) The greatest seasonal differential in temperature between the top and bottom of the concrete slabs was measured in May.

(2) The daily differential is greatest about four P.M.

(3) The differentials occurring when the bottom of the slab is warmer than the top are usually less than when the bottom of the slab is cooler than the top.

(4) The greatest differentials for the case where the bottom of the slab is warmer than the top, usually occurred between four o'clock and eight o'clock in the morning.

(5) The differential in the temperature between the top and middle of a slab is usually much greater than between the middle and bottom of the slab. This difference is more pronounced when the top is warmer than the bottom.

SECTION VII - RESULTS

7.01 Design Factors:

These factors based on three methods of computations, and the test conditions, are given by Tables I-A, for the 20,000 lb. wheel load, II-A for the 37,000 lb. wheel load, and III-A for the 60.000 lb. wheel load. Also given with these factors are the number of coverages at which failure first occurred at the free and doweled end of each slab. Failure in this instance is defined as a crack which creates a separate unit of pavement bounded continuously by the crack itself and the pavement jointings. The capital letters, C, T, and L following the number of coverages given in the last two columns of these tables indicate corner, transverse, and longitudinal cracks respectively. Where no letter is given no cracks were observed at the number of coverages indicated by the table.

The computed stresses and concrete flexural strengths on which these design factors were based are given in Tables I, II, and III. Also given by these tables are the measured thickness "h" of the concrete, and the subgrade or base modulus "k" used in computing the maximum stress in each slab for the given wheel load. The three methods of computation used are defined in paragraphs 5.02, 5.03, 5.04, and 5.05. The primary purpose of these design factors was to check the behavior of the test slabs against the present method of design as outlined in Part IV Chapter XX of the March 1943 edition of the Engineering Manual. These factors are given in the second column of Tables I-A, II-A, and III-A. Columns 3 and 4 of these tables give design factors for alternate methods of computation which are more rigorous mathematically than the present method for interior stresses. The second alternate method which gives design factors based on edge loading is given, as it is more nearly applicable to the conditions where the wheel load approaches and crosses a transverse expansion joint.

7.02 Stationary Wheel Load Deflection Measurements:

The results of these measurements are given on Figures 4.0 to 4.7 inclusive. These figures are bar charts which indicate the deflections graphically for study and comparison. Key diagrams are given on Figures 4.0, 4.1, 4.2, and 4.4 which show the positioning of wheel load and gages. The following table summarizes the contents of the figures showing these results:

Table 7.1

Summary of Content for Figures Giving Results of Stationary Wheel Load Deflection Measurements

Fig. No.	Sections Inclusive From To	Wheel Load in Thousand lbs.	Position of Wheel Load with Reference to Traffic Lane
4.0	A - K	20 & 37	Lanes 2 & 1 Interior
4.1	A - K	20 & 37	Lanes 2 & 1 Edge
4.2	G - L	37 & 60	Lanes 1 & 2 Interior
4.3	M - U	37 & 60	Lanes 1 & 2 Interior
4.4	G - K	37 & 60	Lanes 1 & 2 Edge
4.5	L - U	37 & 60	Lanes 1 & 2 Edge
4.6	A - F & N	37	Lane 1 Interior & Edge
4.7	A - F	60	Lane 2 Interior & Edge

7.03 Moving Wheel Load Deflection Measurements:

The deflections of the concrete pavement under moving wheel loads as recorded at a few of the electrical gage installations are shown in Figures 4.8 to 4.11. The curves shown are called "influence lines of pavement deflection" as they indicate the deflection at one particular point on the concrete pavement as the moving wheel approaches, passes over, and leaves this point. These lines differ from conventional influence lines in that the deflection is measured at a particular point on the structure (slab) as the unit load is in motion across the structure; whereas, a true influence line shows the deflection at a particular point on a structure as a unit load is placed at successive points across the structure. In the latter case, a condition of equilibrium is assumed to exist as the unit load is applied at each point. In the case of the moving wheel load, the shape of the "influence" line depends somewhat on the speed of the traffic which is shown on the figures. In order to compare the maximum deflections and slopes of the "influence" lines of similar pavement designs and joint or interior conditions, the "influence" lines of the different wheel loads have been superimposed. Thus Figure 4.8 shows the deflections of 8-inch slabs on natural subgrade (F1.80 and F2.80) subjected to 37,000 and 20,000 pound wheel loads at the doweled joint, free joint and interior. Figure 4.11 shows the deflections in the interior of the two slabs in the overlay type construction subjected to 60,000 pound wheel load. On each figure a layout of the gages at which deflections were measured is shown.

7.04 Crack Pattern Development and Extent of Failure:

a. Crack Pattern Development: The development of cracks and failure in the pavement of the Test Track is presented in the plotted crack patterns of Figures 5.0 to 5.10 inclusive and the photographs of Figures 7.0 to 7.3 inclusive.

(1) Plotted Crack Patterns: In the case of the original pavement construction, the crack patterns for similar designs of Lanes 1 and 2 are presented together for purposes of comparison. Further comparison is afforded between approximately similar designs in the same lane by selecting the crack patterns which developed at about the same number of coverages for each of the four stages given on the figures. The diagrams to the right of the crack patterns show the magnitude and distribution of the traffic loading. The following tabulation indicates the content of these figures with reference to the section and the approximate number of coverages at which the crack patterns are given for each lane.

Table 7.2

Summary of Traffic Coverages Shown on Crack Pattern Development Figures 5.0 Through 5.10.

Fig. No.	Section	Lane 1					Lane 2				
		Wheel Load in 1000 Lbs.	Number of Coverages (Approx.)				Wheel Load in 1000 Lbs.	Number of Coverages (Approx.)			
			1st Stage	2nd Stage	3rd Stage	4th Stage		1st Stage	2nd Stage	3rd Stage	4th Stage
5.0	A through C	37	18	57	92	226	20	165	335	443	536
5.1	D & E	37	18	57	92	226	20	165	335	443	536
5.2	R through U	37	91	226	493	1063	60	1.5	19	42	133
5.3	F, N & P	37	108	19[illegible]	281	637	60	6	16	32	148
5.4	K, O & Q	37	422	726	986	1486	60	42	80	138	186
5.5	G, H & J	37	499	659	829	1136	60	42	80	138	205
5.6	A	37	661	1921	-	-	-		-	-	-
5.6	A	55	-	-	1970	2202	-	-	-	-	-
5.6	B & C	37	661	1921	-	-	-	-	-	-	-
5.6	B & C	55	-	-	2207	3005	-	-	-	-	-
5.7	D through F	37	661	1921	-	-	-	-	-	-	-
5.7	" "	55	-	-	2207	3005	-	-	-	-	-
5.8	L	37	423	623	343	1493	60	42	80	98	165
5.8	M	37	423	623	343	1493	60	42	98	374	715
5.9	A through C	-	-	-	-	-	60	24	72	98	712
5.10	D through F	-	-	-	-	-	60	24	98	138	712

(2) Photographs of Pavement Failure: The photographs of Figures 7.0 to 7.3 inclusive show the actual condition of selected pavement slabs. Some of the photographs on Figures 7.1, 7.2, and 7.3 show the complete destruction of pavement under traffic. Others, such as the reinforced slabs of G, H, and J in Figure 7.2 and the 6-inch slabs on deep granular fill in Figure 7.1 show a considerable amount of cracking, yet the pavement surface remained in a serviceable condition.

b. Rate of Failure Curves: The rate of slab failure under traffic loading is shown in the graphs of Figures 6.0 to 6.2 inclusive. For each slab these graphs show the number of slab pieces formed by cracks as the traffic coverages are applied. The slabs in each graph are grouped in such a manner that the rates of failure of slabs of similar design subjected to 20,000 and 37,000 pound wheel loads or 37,000 and 60,000 pound wheel loads may be compared. Each figure contains a tabulation which gives the pertinent features for each slab represented by the rate of failure curves.

SECTION VIII - DISCUSSION OF RESULTS

8.01 Evaluation of Design Methods:

The method of design for first consideration is that specified in Part IV, Chapter XX of the March 1943 edition of the Engineering Manual. A design factor of 1.75 is required in the design of airfield pavements which will be subjected to capacity operation. The evaluation of this method of design in the present investigation consists of computing design factors for each pavement slab on the basis of the physical properties and wheel loads involved, and then considering the actual performance of the pavement slabs under the traffic loading. This data is given in Tables I-A, II-A, and III-A in columns 2, 5, and 6. Column 2 gives the design factor by the present method (March 1943 issue of Chapter XX of the Engineering Manual). Columns 5 and 6 indicate the number of traffic coverages at which failure first occurred in each slab at the undoweled and doweled end. In Table I-A the design factors vary from 1.65 to 3.85 for the 20,000 lb. wheel load. Failure occurred in the doweled end and undoweled ends of the 6-inch slabs where design factors varied from 1.65 to 1.90 between 38 and 440 coverages. The lowest number of coverages at which failure occurred at the doweled ends was 78, and the next lowest 348, for the 6-inch slabs under the 20,000 pound wheel load traffic. There was no failure observed at 550 coverages in the 8 and 10-inch slabs which have design factors between 2.34 and 3.85. Comparing similar design factors given in Tables II-A and III-A, for the 37,000 and 60,000 pound wheel loads, it will be noted, that failure occurred earlier at both the doweled and undoweled ends of the slabs than for the 20,000 pound wheel loads where conditions were similar (base and subgrade). This would indicate that the present method of design is slightly optimistic for 20,000 pound wheel loads and more so for 37,000 and 60,000 pound wheel loads.

A similar analysis of the overlay slabs under the 37,000 and 60,000 pound wheel loads will indicate that their design is more adequate when their performance is compared with that of single slabs having similar design factors.

The design factors based on the present method are used throughout the discussion as an index of overload when comparing or evaluating the effect of variation in subgrades and bases.

The design factors given for the interior and edge of the slabs by the alternate method (Columns 3 and 4 Tables I-A, II-A, and III-A) are presented, as they are an improvement mathematically on the present method, and also analyze two positions of load, (interior and edge) instead of one. In practically every case, failure of the slabs occurred first at the undoweled or free end and second at the doweled end; therefore, it may be argued that the design of pavements should be based on a consideration of both interior and edge loads. The degree of influence to allow the stress determined for the edge condition of load should be controlled by the efficiency of the load transfer method across the joints; i.e., for a free transverse or free longitudinal expansion joint, the design should be based entirely on consideration of edge loading.

It is for note that the design factors given in Tables I-A, II-A, and III-A are based on the flexural strength of the concrete at the time of test. Complete conformance with the present method (Chapter XX, March 1943 edition of the Engineering Manual) would require that the 28-day flexural strength of the concrete be used. However, the comparison with the service behavior of the slabs is more informative if physical properties of bases, subgrades, and concrete at the time of test are used in computing the design factors. The difference in design factors for the 28-day flexural strengths of the concrete may be obtained by using the estimated 28-day strengths given in paragraph 4.04 Table 4.4. This will in no way alter the general conclusions drawn in the foregoing discussion and summarized below.

The following conclusions have been discussed and may be summarized as follows:

a. The tentative design curves given in Part IV Chapter XX of the Engineering Manual, March 1943 edition, are not conservative for wheel load traffic of 37,000 and 60,000 pounds, while those for 20,000 pound wheel loads are more nearly correct.

b. The results of the tests and theoretical computations both indicate that stresses at the corners and edges of a concrete pavement slab control its design.

c. The tentative design criterion for reference (19), will yield more adequate designs for 37,000 and 60,000 pound wheel loads than will the tentative design curves for single slabs given in Part IV Chapter XX of the 1943 edition of the Engineering Manual.

8.02 Evaluation of the Effect of Variation in Types of Subgrade:

The types of subgrade tested with and without base course include the natural plastic clay subgrade which had "k" values at the time of the traffic tests varying from approximately 70 to 150 $lbs/in.^3$ (See Tables I to III inclusive) and the specially constructed deep granular subgrades (sections R, S, T, and U) having "k" values ranging from about 200 to 400 $lbs/in.^3$.

This study of the effect of variation in types of subgrade on the service behavior of concrete pavements is complicated by the variation in subgrade moduli as indicated in the preceding paragraph. The study is further limited by the fact that only 6-inch plain concrete slabs were tested on the deep granular subgrades. These limitations, along with other variable test conditions such as variations in moisture content, actual pavement thicknesses, quality of concrete and boundary conditions, should be considered when analyzing the traffic test data and observations which are discussed in the following paragraphs.

The 6-inch slab on natural subgrade, the 6-inch slabs on 6-inch bases, the 8-inch slab on natural subgrade, and the 8-inch slab on

a 6-inch base were either completely destroyed or so badly cracked by approximately 370 coverages of the 37,000 pound wheel load that replacement was necessary before traffic could be continued. The condition of these test slabs is illustrated by the photographs in Figure 7.1, and by the crack patterns shown in Figures 5.0, 5.1, and 5.3. In contrast to the service behavior of this group of test slabs, the 6-inch slabs on the deep granular subgrades developed cracks at a much slower rate; and, although badly cracked, remained in service for the full 2285 coverages of the 37,000 pound wheel load. The photographs in Figure 7.1 show the condition of these slabs at approximately 370 coverages and at 1400 coverages. The crack pattern development of the 6-inch slabs on the prepared deep granular subgrade under the 37,000 and 60,000 pound wheel load traffic is shown in Figure 5.2. Under traffic of the 60,000 pound wheel load in Lane 2, the 6-inch slabs on deep granular subgrade cracked badly but remained in service for the total 712 coverages. The crack development at 24, 80, and 158 coverages is shown by the photographs in Figure 7.0. The absence of spalling along cracks and joints in the 6-inch slabs on deep granular subgrades, under both 37,000 and 60,000 pound wheel load traffic, is shown by the photographs in Figures 7.0 and 7.1.

The superior service behavior of the 6-inch slabs on deep granular subgrades under 37,000 and 60,000 pound wheel load traffic was predictable from the relatively small deflections measured at the transverse and longitudinal joints. These deflection measurements are presented graphically as bar charts in Figures 4.3 and 4.5.

The conclusions indicated by the test results and preceding discussion may be summarized as follows:

a. The rate of failure of the 6-inch slabs under the action of the 37,000 pound wheel load traffic varied greatly, and was least for the slabs on the deep granular subgrades.

b. Under the 60,000 pound wheel load traffic, the 6-inch slabs on the deep granular subgrades, although badly cracked, gave better service behavior than the 8 and 10-inch plain concrete slabs on the natural plastic subgrade.

c. The better service behavior of the 6-inch slabs on deep granular subgrades under both 37,000 and 60,000 pound wheel load traffic was predictable from the relatively small deflections measured under stationary wheel loads.

8.03 Evaluation of the Effect of Variation in Types and Thickness of Granular Base Courses:

Granular non-cohesive base course materials studied at Lockbourne No. 1 included: bank run sand and gravel from local glacial deposits, sand from local gravel plant flume deposit, and crushed limestone from a local quarry. The study of all three base course materials is limited to the data and observations obtained from 20,000 and

37,000 pound wheel load accelerated traffic tests on 6-inch plain concrete slabs constructed on a 6-inch thickness of base. The study of the sand and gravel base material includes 6 and 12-inch base thicknesses with 8 and 10-inch plain concrete slabs tested under 37,000 and 60,000 pound wheel load traffic. The 6-inch plain concrete slabs with 6-inch granular bases under the 37,000 pound wheel load traffic include slabs B1.66L, C1.66S, D1.66, and E1.66M, having design factors varying from 1.22 to 1.43. These slabs were completely destroyed in less than 300 traffic coverages, indicating excessive overloading by the 37,000 pound wheel load. The 6-inch slab on a 6-inch crushed stone base was the only slab that showed the effect of type of base on its service behavior. This slab had a lower rate of failure than the 6-inch slab of compacted sand and gravel. Comparison of the other types of base materials appear to have been vitiated by excessive overloading. (See the crack pattern development, Figures 5.0 and 5.1, and the rate of failure curves in Figure 6.0). Photographs in Figure 7.1 show the complete destruction of the 6-inch slabs on 6-inch bases under the 37,000 pound wheel load traffic. The photographs, in the same Figure, illustrate a very interesting and significant comparison between a 6-inch slab on a natural clay subgrade and the 6-inch slabs on 6-inch granular bases, which have been subjected to excessive overloading. The slab shown in the photograph is slab A1.60 which is a 6-inch slab constructed on natural clay subgrade having "k" values varying from 76 to 139 lbs/in.^{3}. Giving due consideration to the variation in subgrade moduli, the comparison of service behavior indicates that 6 inches of granular non-cohesive base course, regardless of type, under plain concrete pavements subjected to excessive overloading, is inadequate as it allows excessive shear deformation once failure of the slab begins. Furthermore, the presence of thin granular non-cohesive base courses on low bearing subgrades accelerates shear deformation, once the slab is cracked and is continued to be subjected to overload.

Under the 20,000 pound wheel load traffic on slabs B2.66L, C2.66S, D2.66, and E2.66M, which had design factors varying from 1.65 to 1.85, the relative value of the base courses is more accurately evaluated. A total of 550 coverages of the 20,000 pound wheel load traffic produced no failure at the doweled ends of the 6-inch slab on compacted sand and gravel and the 6-inch slab on 6 inches of crushed stone. (See crack pattern development, Figures 5.0 and 5.1). The rate of failure of the slab on loose sand and gravel was less than that on compacted sand base. The rate of failure curves for this group of slabs is shown in Figure 6.0. It should be noted that the rate of failure of the 6-inch slab on natural subgrade (slab A2.60) is less than that for the slabs on loose sand and gravel and on sand bases. This may be considered as additional evidence for the argument against the use of thin granular non-cohesive bases on clay type subgrades with concrete pavements that are likely to be overloaded.

In evaluating the effect of thickness of granular base courses, the degree of overloading controls. The rate of failure curves in Figure 6.1 show that, for the 8-inch slabs on 6 and 12-inch bases, the rate

of failure was materially reduced for the slab with the 12-inch base under traffic at 37,000 pound wheel load where design factors varied from 1.73 to 1.98; while for these same designs, there was no reduction in the rate of failure under the 60,000 pound wheel load traffic where design factors were 1.25 and 1.56. The beneficial effect of a thicker base (12 inches versus 6 inches) is evident from the crack pattern development, Figure 5.4, and the rate of failure curves, Figure 8.1, for the 10-inch slabs under the 37,000 pound wheel loading where design factors of 2.60 and 2.74 are indicated. This is also true but to a lesser degree for the 10-inch slabs under the 60,000 pound wheel load traffic where design factors are 2.02 and 1.66. An increase in base thickness from 6 to 12-inches on the prepared deep granular subgrades reduces the rate of failure of 6-inch slabs under both the 37,000 and 60,000 pound wheel loadings, where design factors are 2.18, 2.10, 1.54, and 1.56 respectively.

The following conclusions are presented as a summary of the preceding discussion:

a. Crushed limestone and bank-run sand and gravel base materials compacted to a 6-inch thickness effected a reduction in the rate of failure in 6-inch plain concrete pavements, whereas, for the same degree of overload, a 6-inch compacted sand base and a 6-inch loose sand and gravel base yielded no benefit.

b. The 12-inch thickness of compacted bank-run sand and gravel, as compared with a 6-inch thickness, effected lower rates of failure for the 6, 8, and 10-inch slabs when the design factor was greater than 1.66. When this factor was less than 1.66 (60,000 pound wheel loading on 6 and 8-inch slabs) the greater thickness of base course showed no benefit.

c. Where overload is anticipated the inclusion of a granular non-cohesive base course is not desirable.

8.04 Evaluation of the Effect of Joint Type and Spacing:

The types of joints studied by this investigation and their spacing are defined in paragraph 2.01.

The condition of the joints, as affected by moisture and temperature conditions prior to and during the time of testing, is very pertinent in considering the results of the pre-traffic deflection measurements and the subsequent behavior of the joints under traffic loading. The test slabs were constructed and the joints were sealed in October 1943 during a period of unusually dry weather. While the weather was still dry, the finished pavement was covered with straw and tarpaulins to protect the concrete and foundation against freezing temperatures. This cover remained on the pavement until the middle of March 1944, at which time preparations were commenced for conducting the tests. All joint seals were in good condition at the start of testing, and they re-

mained in good condition until pavement failures occurred under traffic loading. The major portion of the testing was conducted under favorable weather conditions, with air temperatures considerable higher than at the time of construction. Furthermore, during the period of heaviest precipitation, prior to testing, (see weather charts, Figures 3.0 and 3.1) the test pavements were subjected to no loading that would cause joint action and vitiate the joint seal. These contributing factors, along with the fact that adequate surface and subsurface drainage was provided, make it reasonable to assume that there was little accumulation of moisture in the subgrade, due to leaky joints, until after the pavements had been subjected to considerable traffic loading. A limited number of moisture determinations of the subgrade, taken during installation of the electrical deflection gage assemblies, showed no increase in moisture content at the joints.

Information on the behavior of joints was obtained from the stationary wheel load deflection measurements and from a limited number of moving wheel load deflection measurements. For ready comparison, the stationary wheel load deflection measurements are presented graphically as bar charts in Figures 4.0 to 4.7 inclusive. Typical moving wheel load deflection measurements are shown in Figures 4.8 to 4.10 inclusive. These data are supplemented by observations of the crack pattern development made during the traffic tests and presented graphically in Figures 5.0 to 5.10 inclusive. For the purpose of discussion, the joints are grouped according to their principal intended purpose, such as "expansion", and "contraction".

Expansion Joints: The expansion joint was used principally as a transverse doweled or undoweled joint between the transition slabs and the various test slabs as shown in the plan of the Test Track, Figure 1.[illegible] The joint was used longitudinally in only one case which was as a free joint between Lanes 1 and 2 in the reconstructed slabs of the north tangent (see Figure 1.5). Examination of the deflection charts in Figures 4.0 through 4.7 shows that, in almost every case, the undoweled transverse expansion joint permits the largest deflections and has the poorest efficiency. Joint efficiency, as used in this report, is the ability of the joint to transfer deflection when the load is entirely on one side during its initial approach, and is expressed by a percentage which is twice the deflection on the opposite or unloaded edge divided by the sum of the deflection of both edges times 100. Efficiencies of undoweled expansion joints ranged from 14 to 86 percent for the 37,000 pound wheel load and from 4 to 58 percent for the 60,000 pound wheel load. The free expansion joint was the weakest part of the pavement and in several designs, where overloading was excessive, failures occurred at this point under the initial loading for measuring deflection and before traffic was started. With very few exceptions, this joint was the starting point for progressive failure of the test slab. This is apparent from the crack development shown in Figures 5.0 to 5.10 inclusive. The two free joints showing the highest efficiencies (57 and 58 percent) were the lapped joints in the overlay slabs D2.7-66 and E2.7-66M where

the joints in the top slabs were lapped 16 inches over their counterparts in the base slabs. The deflections produced at these joints under the 60,000 pound wheel load are comparable to those produced at the doweled joints (see Figure 4.7). The doweled transverse expansion joints permitted less deflection than the free joints, and had better efficiencies. The efficiencies for these joints varied from 82 to 100 percent for the 37,000 pound wheel load, and from 51 to 99 percent for the 60,000 pound wheel load. The two expansion joints employing heavier dowels at closer spacing (1-1/2" ∅ smooth bars, 16" long, 8" O.C.) gave superior performance under both stationary and moving wheel loads. The stationary wheel load deflection measurements, shown in Figure 4.6 were smaller for the heavy dowels, and the indicated efficiencies varied from 90 to 98 percent. The better performance under moving wheel loads was indicated by the crack patterns in Figure 5.7. The two transverse expansion joints, with the experimental offset dowels in slabs H1.8R-0 and J1.8R-0, indicated better efficiency when compared with the performance of the common dowels in the other 8-inch slabs under the 20,000, 37,000 and 60,000 pound wheel loadings.

The stationary wheel load deflection measurements for the slabs in sections R, S, T, and U (see Figures 4.3 and 4.5) showed that the deflections at transverse undoweled and doweled expansion joints were smallest for the pavements on high bearing subgrades. The deflection charts also indicated that 6 inches of base course was ineffective in reducing the magnitude of the deflections at expansion joints in 8 and 10-inch slabs, whereas 12 inches of base course reduced the magnitude of the deflections.

Contraction Joints: Joints of this type used in the Test Track included the dummy ribbon joint and the keyed and doweled construction joints. The test results indicated in general, that the contraction joints were superior to expansion joints in their performance under stationary and moving wheel loads. The pre-traffic deflection measurements, presented graphically in Figures 4.0, 4.2, 4.3, and 4.7, show the magnitude of the deflections at dummy ribbon contraction joints for 20,000 and 60,000 pound stationary wheel loads. These charts show that the magnitude of deflection at dummy joints was about equal to or slightly less than those measured at the doweled expansion joints. The efficiencies of the dummy joints were consistently better than for the doweled expansion joints. Under the 20,000 pound stationary wheel load (Figure 4.0), efficiencies at seven dummy joints vary from 90 to 100 percent with an average of 96 percent. Measurements taken after traffic at three joints where failure had not occurred, showed efficiencies varying from 86 to 98 percent with an average of 91 percent, indicating the expected reduction in efficiency with traffic Under the 60,000 pound stationary wheel load (Figures 4.2, 4.3, and 4.7) efficiencies from deflection measurements at seventeen dummy contraction joints, varied from 80 to 98 percent with an average of 91 percent. It should be remembered that these joint efficiencies were determined from deflections measured at joints that have not been subjected to traffic

loading and at a time when the joints were tightly closed by temperature effects. The behavior of the dummy contraction joints under traffic loading is shown by the crack pattern development plots in Figures 5.0 to 5.10 inclusive. The crack patterns show that initial failures usually occurred in the vicinity of the expansion joints before they developed along the transverse and longitudinal dummy joints. However, the corner and edges formed by the dummy joints definitely influenced the position of initial failure. Very frequently, initial failure occurred at the corners formed by the intersection of an expansion joint and the longitudinal dummy joint. The smaller slab units, formed by the transverse and longitudinal dummy joints, were less stable under the action of all traffic loadings than the larger slabs. This was observed to be the case regardless of whether the slab was bounded by free or doweled transverse expansion joints. Also, in a few instances where moving wheel load deflections were measured, the deflections of the smaller transition slabs were greater, and a tendency to tilt was observed under the 37,000 and 60,000 pound wheel loadings.

The performance of the longitudinal keyed construction joint under stationary wheel loading is limited to the 37,000 pound wheel load. Deflections measured at the keyed joint under this wheel load were smaller than at a doweled transverse expansion joint, and the efficiencies were comparable except that they were somewhat more variable (67 to 100 percent). Evaluation of the behavior of the keyed construction joint under traffic loading was penalized by the pattern of traffic which first routed 37,000 pound wheel load traffic on the Lane 1 edge, followed by 60,000 pound wheel load traffic on the opposite edge. The concentration of 37,000 pound wheel load traffic in Lane 1 broke the top of the groove portion of the joint. This was indicated by the crack patterns in Figures 5.0 to 5.10 inclusive. In like manner, the 60,000 pound wheel load traffic in Lane 2 broke the bottom of the groove portion of the joint. This was observed when certain slabs in Lane 2 were removed for replacement.

The longitudinal doweled butt-type construction joint in sections G, H, and J indicated more uniform deflection and better effeciencies than the longitudinal keyed construction joint in slabs of equal thickness (Slabs F1.80, H1.86, and P1.812). Performance of the doweled construction joint under traffic loading was superior to the keyed construction and dummy contraction joints. Crack patterns in Figure 5.5 show no cracks that can be directly attributed to failure of the doweled construction joint.

While the test results and preceding discussion have presented several interesting indications of the functioning of joints of different designs, it is believed that they should be treated as such and their substantiation or disproval should await the developments of future tests or observations on actual airfield pavements. However, the following conclusions and indications are presented in summary:

(1) Load transfer devices of one type or another with transverse expansion joints, are necessary to the structural integrity of rigid pavement designs.

(2) The tests indicated that slab units, as small as 10' x 10' x 10" thick or less and on plastic subgrade even though tightly jointed, are undesirable for rigid pavements when the wheel load is 37,000 pounds or greater.

(3) It was indicated that construction joints should be doweled instead of keyed for wheel loads of 37,000 pounds and greater, when traffic wheel loading can be concentrated along the joint.

8.05 <u>Evaluation of the Effect of Steel Reinforcement in Concrete Pavement Slabs:</u>

The study of steel reinforcement as used in concrete pavement slabs, is limited to the data and observations obtained from 37,000 and 60,000 pound wheel load traffic tests made on three 8-inch, wire mesh reinforced, original slabs placed on natural plastic subgrade and six 8 and 10-inch slabs heavily reinforced with wire mesh or bar mat and placed on 6 inches of compacted sand and gravel base course on approximately 24 inches of remolded subgrade, and to any of the unreinforced slabs with which these might be compared. The latter reinforced group of slabs form a portion of the reconstruction necessitated after approximately 370 coverages of the 37,000 pound wheel load traffic. The nine designs with pertinent details of their reinforcing are listed in Table 2.3 paragraph 2.02 e. The 10-inch plain concrete slab A1.106 was included in the listing, since it was one of the group of replacement slabs and was comparable to the 10-inch reinforced slabs.

Results of traffic loading tests on the reinforced slabs and any inferences or conclusions drawn therefrom are subject to the variables and limitations of transition and jointing which was not comparable in all cases to the plain concrete slabs. These variables and limitations of jointing will be reviewed before proceding with the discussion. Referring to Figure 1.1, it is observed that there are no transition slabs between the slabs in sections G, H, and J, and that the transition slab between sections J and K is only 5 feet wide instead of the usual 10 feet where transition slabs were present. As to the jointing, slabs in sections G, H, and J were separated transversely by expansion joints and longitudinally by doweled construction joints. Transverse expansion joints at each end of slabs J1, and J2.8R-0 had a special offset dowel (see detail of joint in Figure 1.3). Expansion joints between the slabs in sections G and H were doweled with the conventional 1-inch round dowels, 16 inches long, on 12-inch centers. The expansion joint at the west end of section G was undoweled. Furthermore, slabs in section J have no dummy contraction joints, and those in sections G and H have only longitudinal dummy contraction joints, with the exception of slab H1.8R-0 which had both longitudinal and transverse dummy joints. Replacement slabs of the reconstructed section had certain features of

jointing and transition which were not comparable to the plain and reinforced slabs of the original construction. These slabs (see Figure 1.5) were separated from adjoining transition slabs by transverse expansion joints which were undoweled at one end and doweled at the other end. The doweled joints had the conventional 1-inch round dowels, 16 inches long, on 12-inch centers, with the exception of two joints joining slabs E1.8R-6 and F1.8R-6 to their common transition, which had 1.5 inch round dowels, 16 inches long, on 8-inch centers. The undoweled or free joints had prefabricated bituminous joint filler, whereas similar joints in the original construction employed redwood filler. The slabs were separated from the overlays in Lane 2 by a longitudinal undoweled expansion joint which had redwood filler for the 10-inch slabs and prefabricated bituminous filler for the 8-inch slabs. The longitudinal joint for slab N1.8R-6 was the original keyed construction joint. All reinforced slabs and the one 10-inch plain concrete slab A1.106 of this group were constructed without dummy contraction joints. Furthermore, the 10-foot transition slabs between slabs B1.10R-6, C1.10R-6, D1.8R-6, E1.8R-6, and F1.8R-6 were heavily reinforced with bar mats top and bottom. The transition slab between slabs A1.106 and B1.10R-6 had bar mat reinforcing top and bottom, in the east half adjacent to B1.10R-6. No attempt will be made to evaluate the effect of these variables of transition and jointing; but they should be considered in determining the validity of the test results and in qualifying the indications and conclusions presented in the paragraphs that follow. The presence of the ramp at slab A1.106 perhaps contributed in a slight degree, to the disintegration of this slab, and may also be considered a factor particularly since the joint between slab and ramp was an undivided joint.

The crack pattern development plots, shown in Figures 5.3 and 5.5, indicate that after 390 coverages of the 37,000 pound wheel load traffic, the 8-inch reinforced slabs on natural subgrade, having design factors varying from 1.36 to 1.30 showed only slight distress, whereas the 8-inch plain concrete slab on natural subgrade (slab F1.90) was completely destroyed. Similar slabs on compacted sand and gravel bases, 8 and 12 inches in thickness, were either badly cracked (slab N1.86) or failed at the undoweled transverse expansion joint (slab P1.812). This comparison indicated that steel reinforcing in concrete pavement delayed the early development of structural failures providing the degree of overload was not excessive. When overloading was excessive, as it was for the 60,000 pound wheel loading on 8-inch pavements (see design factors in Table III), the reinforcing was less effective in retarding early breakup. This was determined by inspection of the crack patterns in Figure 5.5, which showed that, at only 42 coverages of the 60,000 pound wheel load traffic, test slabs G2.8R-0, H2.8R-0, and J2.8R-0 had as many or more failures than slabs G1.8R-0, H1.8R-0, and J1.8R-0 had at 500 coverages of the 37,000 pound wheel load traffic. An early initial failure under the 60,000 pound wheel load traffic was indicated by the relative magnitude of the stationary wheel load deflections shown in Figures 4.2 and 4.4, which were taken prior to traffic. The rapid rate of failure of the reinforced slabs under 60,000 pound wheel load traffic as compared to the 37,000 pound wheel loading, is shown by the rate of

failure curves in Figure 6.1. Three stages in the crack pattern development under 60,000 pound wheel load traffic are exhibited by the photographs in Figure 7.2. The third stage includes photographs of two 8-inch plain concrete slabs (slabs N2.86 and P2.812), and provides a striking comparison of the service behavior of plain and reinforced pavements under equivalent traffic loading. This comparison serves to demonstrate the beneficial effect of light reinforcing in holding the broken concrete of an overloaded pavement slab together, and thus prolonging its serviceability. Data in Figure 6.1 indicate that the weight of reinforcing was effective in reducing the rate of failure of the 8-inch concrete slabs when design factors were not less than 1.68 (37,000 pound wheel load), and was only slightly effective when design factors did not exceed 1.41 (60,000 pound wheel load).

The heavily reinforced 8 and 10-inch reconstructed slabs in Lane 1 of the Test Track (see Table 4.3 paragraph 2.02 e.) were subjected to approximately 1920 coverages of the 37,000 pound wheel load traffic during the periods 19 July to 6 September, and 29 November to 9 December 1944. This loading produced corner failures in the 8 and 10-inch pavements reinforced with 68 pound wire mesh in the top and bottom (slabs D1.8R-6 and B1.10R-6) and in the 8-inch slab reinforced with 1/2 inch bar mat, 8-inch centers, top and bottom (slab F1.8R-6). The 8 and 10-inch slabs reinforced with 156 pound wire mesh in the top and bottom (slabs E1.8R-6 and C1.10R-6) and the 8-inch slab reinforced with 1/2 inch bar mat, 6-inch centers, top only (slab N1.8R-6) showed only slight hair cracking after about 1900 coverages of the 37,000 pound wheel load traffic. Approximately ten months later, during the period 17 September to 18 October 1945, the same group of slabs, with exception of slab N1.8R-6, received an additional 1084 coverages of 55,000 pound wheel load traffic. This additional traffic loading produced failures in the 8 and 10-inch slabs reinforced with 156 pound wire mesh and caused additional distress in the other 8 and 10-inch slabs of this group. An examination of the data in Figure 6.2 will show that rate of failure of the 8-inch slab with 68 pound wire mesh top and bottom (slab D1.8R-6) increased appreciably under the 55,000 pound wheel load traffic, whereas the plots for the other 8 and 10-inch slabs of this group show only a slight increase. It should also be noted that the 10-inch plain concrete slab A1.106, which is comparable to the 10-inch reinforced slabs, showed a decided increase in rate of failure under the 55,000 pound wheel load traffic. The photographs in Figure 7.3 show the extent of failure in the test slabs after approximately 760 coverages of the 37,000 pound wheel load and also the final condition of the pavement following all 37,000 and 55,000 pound wheel load traffic. Comparison of the service behavior of the 10-inch plain concrete slab A1.106 with the 8 and 10-inch reinforced slabs, shown by the photographs in Figure 7.3 emphasizes the advantage of steel reinforcing in prolonging the serviceability of overloaded concrete pavements. Examination of the 37,000 pound stationary wheel load deflection measurements, shown by the charts in Figure 4.6, indicates that reinforcing is effective in reducing the magnitude of the deflections at points of overload, such as the edge at

an undoweled expansion joint, and corners formed by the intersection of undoweled longitudinal and transverse expansion joints.

To summarize the preceding discussion, the following conclusions are presented:

a. Performance of concrete pavement, designed with reinforcing steel, is improved and serviceability is prolonged under traffic loading, particularly when pavement is overloaded.

b. Benefits derived from the inclusion of reinforcing steel in overloaded concrete pavements increase with the amount of steel present.

c. Under excessive overload, the effect of different amounts of reinforcing steel is lost. The steel also loses some of its power to prolong pavement life.

d. Assuming that subgrade conditions were equal for both reinforced and unreinforced pavement slabs, results of the stationary wheel load deflection measurements indicate that reinforcing was effective in reducing the magnitude of the pavement deflections at points of overload such as free edges and corners. (See Figure 4.6)

8.06 <u>Evaluation of the Behavior of Concrete Overlay Slabs on Unbroken and Broken Base Slabs</u>:

The pertinent design features of the overlay slabs tested under 37,000 and 60,000 pound wheel load traffic are summarized in Table 2.4 of paragraph 2.02 f.

The transition and jointing details of the overlay designs are shown in Figures 1.1 and 1.5; and it should be noted that these details are not comparable in every case. The 5 and 7-inch top overlay slabs on unbroken base pavements in sections L and M (see Figure 1.1) were bounded by undoweled transverse expansion joints, a longitudinal keyed construction joint and free edges with one 10-foot transition slab between section L and the east turn. The 7-inch top slabs on the preloaded base slabs (slabs A2.7-60 through F2.7-80, Figure 1.5) were bounded by doweled and undoweled transverse expansion joints, a free longitudinal expansion joint, and free edges with 10-foot transition slabs adjacent to the ends of each test slab. The undoweled transverse expansion joints between slabs D2.7-66, E2.7-66M, and their common transition slab were offset 1.5 feet from the joint in the base pavement. The transition slab at each end of the series of overlays was constructed as a ramp to carry the loading equipment from the adjoining pavement onto the overlays. All ten overlay designs had both transverse and longitudinal dummy ribbon joints. The variation in jointing and boundary conditions limits the comparisons of overlay slabs to those test slabs having similar jointing and boundary conditions. These limitations are important in analyzing and discussing the test results which include stationary wheel load deflection measurements, crack pattern development, and rate of failure.

The stationary wheel load deflection measurements taken prior to traffic loading are presented as bar charts in Figures 4.2, 4.3, and 4.5 for the slabs in sections L and M, and in Figure 4.7 for top overlay slabs on 3/4-inch sand-asphalt cushion, and 6-inch unbroken base pavements under the 37,000 and 60,000 pound stationary wheel loadings are shown in Figures 4.2, 4.3, and 4.5. Examination of these deflection measurements show that the increase in magnitude of deflection is approximately proportional to the increase in wheel loading. Comparison of interior deflections for overlays and single slab designs indicates that the deflections of the 5-inch overlays (section L) and deflections of the 8-inch plain and reinforced slabs on natural subgrade and base (see Figures 4.0, 4.2, and 4.3) are approximately of the same order of magnitude. Likewise, Figures 4.2 and 4.3 show that the magnitude of deflections of the 7-inch overlays (section M) compares with that for the 10-inch plain concrete slabs on natural subgrade and base. The above comparison for the 5-inch top overlay slabs holds for the 7-inch top overlay slabs on broken base slabs with a 3/4-inch minimum sand-asphalt cushion (slabs A2.7-60, B2.7-66L, and C2.7-66S, see Figure 4.7). The interior deflections of the 7-inch top slabs placed directly on the base slabs (slabs D2.7-66, E2.7-66M, and F2.7-80) are comparable with the 10-inch plain concrete slabs on natural subgrade and base. The above comparisons are inconsistent for edge and corner load positions where boundary conditions have more effect. The magnitude of the deflections of the top slabs on sand-asphalt cushion is greater than that of the top slabs placed directly on the base slab (see Figure 4.7). Electrical deflection measurements taken simultaneously in the top and bottom slabs of the overlay designs F2.7-80 and M2.7-60 showed that the deflections of the top slabs and base slabs were very nearly equal (see Figure 4.11). This was true for both designs with no cushion course and with 3/4 inches of sand-asphalt. The influence curves of deflection in Figure 4.11 indicate that the overlay with no cushion course recovered more rapidly than the slab on 3/4 inches of sand-asphalt cushion.

The crack pattern development of the overlay designs under traffic loading is presented in Figure 5.8 for sections L and M, Figure 5.9 for slabs A2.7-60, B2.7-66L, and C2.7-66S, and in Figure 5.10 for slabs D2.7-66, E2.7-66M, and F2.7-80. These crack patterns, which were reproduced from the original field plots, were carefully selected to show the development of failure, and for the overlays of sections L and M (Figure 5.8), to show the comparable service behavior of the slabs under 37,000 and 60,000 pound wheel load traffic. The rate of failure curves, which were prepared from the crack pattern records, are shown in Figure 6.2. The comparative service behavior of the overlay designs under 37,000 and 60,000 pound wheel load traffic is evident from an analysis of these test results. However, there are certain significant characteristics and additional observations not indicated by the results that are worthy of mention. The crack development in the overlays was similar to that for single slab design in that failures occurred first as corner breaks and longitudinal cracks at the undoweled transverse expansion joints, and then in a similar manner at the dummy contraction joints. As the stresses are relieved at these joints, maximum stresses shift to other parts of the slab causing progressive failure.

It is interesting to observe that under both 37,000 and 60,000 pound wheel load traffic the 7-inch top slabs on unbroken base slabs (M.7-60 and M2.7-60) showed very little progressive failure and little cracking along dummy contraction joints. The better service behavior of these two test slabs may be due to any one or a combination of influencing factors such as actual thickness and quality of concrete, and the better condition of the sand-asphalt cushion course. The crack pattern development and rate of failure curves, together with observations made during the testing, indicated that with the exception of two 7-inch top slabs in section M, the sand-asphalt cushion course penalized the structural behavior of the overlay designs. This was due to insufficient curing of the sand-asphalt cushion material prior to placing the overlay concrete. During the traffic tests, asphalt from the cushion course was observed pumping through the contraction joints and cracks, and later when the broken concrete was removed, the sand-asphalt cushion material was found to be of soft consistency. This set of circumstances should not preclude the use of a cushion or leveling course but it does warrant the conclusion that the bituminous cushion material should be thoroughly cured before placing the overlay concrete. The broken overlay slab L2.5-60 and the 3/4-inch sand-asphalt cushion course were removed following the 60,000 pound wheel load traffic. Examination of the base slab, which was unbroken prior to placement of the top slab revealed a crack pattern which coincided with that for the top slab. In the case of top slabs on broken base slabs, the crack pattern development in the top slab was defined initially by the original pattern of cracks in the base slab. This indicated that the position of maximum stress in the top slab was influenced by the cracks or planes of weakness in the base slab. However, this was not the case for an 18-inch lap of a top slab over an undoweled transverse expansion joint. This condition was present for the transition overlay between slabs D2.7-66 and E2.7-66M (see Figure 1.5). The crack pattern development for these slabs, shown in Figure 5.10, indicated no detrimental effects due to the 18-inch overlap of the free expansion joints.

Test results and the preceding discussion indicate certain conclusions which are summarized as follows:

(1) Where no initial cracks are present in the base slab of an overlay, the crack patterns produced by traffic overloading are essentially the same for the top and bottom slabs.

(2) When overlays on broken base slabs are overloaded, the original crack pattern of the base slab influences the pattern developed in the top slabs.

(3) Where sand-asphalt leveling or cushion courses are used, every precaution should be taken to insure complete curing of the material before placing the overlay concrete.

(4) For the two cases tested, no detrimental effect was produced by a lap of 18 inches of a top slab over a free joint in the base pavement.

(5) When both overlay and single slab designs are overloaded, the service behavior of the overlay slabs is better under the action of 37,000 and 60,000 pound wheel load traffic even though the design factor based on present design methods is the same or somewhat better for the single slabs.

SECTION IX - CONCLUSIONS

9.01 Evaluation of Design Methods:

a. The tentative design curves given in Part IV Chapter XX of the Engineering Manual, March 1943 edition, are not conservative for wheel load traffic of 37,000 and 60,000 pounds, while those for 20,000 pound wheel loads are more nearly correct.

b. The results of tests and theoretical computations both indicate that stresses at the corners and edges of a concrete pavement slab control its design.

c. The tentative design criterion for "Portland Cement Overlay Pavements", issued June 1945, by the Office of Chief of Engineers, will yield more adequate designs for 37,000 and 60,000 pound wheel loads than will the tentative design curves for single slabs given in Part IV, Chapter XX of the March 1943 edition of the Engineering Manual.

9.02 Evaluation of the Effect of Variation in Types of Subgrade:

a. Rate of failure of 6-inch slabs under the action of the 37,000 pound wheel load traffic varied somewhat. Slabs on the deep granular subgrades were found to be the most resistant to the action of the 37,000 pound wheel load.

b. Under the 60,000 pound wheel load traffic, the 6-inch slabs on the deep granular subgrades, although badly cracked, remained serviceable longer than the 8 and 10-inch plain concrete slabs on the natural plastic subgrade.

c. The better service behavior of the 6-inch slabs on deep granular subgrades, under both 37,000 and 60,000 pound wheel load traffic was predictable from the relatively small deflections measured under stationary wheel loads.

9.03 Evaluation of the Effect of Variation in Types and Thickness of Granular Base Course:

a. Crushed limestone and bank-run sand and gravel base materials compacted to a 6-inch thickness effected a reduction in the rate of failure in 6-inch plain concrete pavements where the design factor was greater than or equal to 1.65; whereas, for the same degree of overload, a 6-inch compacted sand base and a 6-inch loose sand and gravel base yielded no benefit.

b. The 12-inch thickness of compacted bank-run sand and gravel, as compared with a 6-inch thickness, effected lower rates of failure for the 8 and 10-inch slabs when the design factor was greater than 1.66. When this factor was less than 1.66 (60,000 pound wheel loading on 6 and 8-inch slabs) the greater thickness of base course showed no benefits.

c. Where overload is anticipated the inclusion of a granular non-cohesive base course is not desirable.

9.04 Evaluation of the Effect of Joint Type and Spacing:

a. Load transfer devices of one type or another, in transverse expansion joints, are necessary to the structural integrity of rigid pavement designs.

b. The tests indicated that slab units, as small as 10' x 10', and up to 10" in thickness or less, even though tightly jointed, are undesirable for rigid pavements when the wheel load is 37,000 pounds or greater.

c. It is indicated that construction joints should be doweled instead of keyed for wheel loads of 37,000 pounds and greater, when traffic wheel loading can be concentrated along the joint.

9.05 Evaluation of the Effect of Steel Reinforcement in Concrete Pavement Slabs:

a. Steel reinforcement in concrete pavements prolongs their serviceable life under traffic loading, especially when pavements become overloaded.

b. The benefits resulting from the inclusion of reinforcement in overloaded concrete pavements increase with the amount of steel present.

c. When overload becomes excessive the effect of different amounts of reinforcement is lost, and the steel also loses some of its power to prolong pavement life.

d. Assuming that subgrade conditions were equal for both reinforced and unreinforced pavement slabs, results of the stationary wheel load deflection measurements indicated that reinforcing was effective in reducing the magnitude of the pavement deflections at points of overload such as free edges and corners. (See Figure 4.5).

9.06 Evaluation of the Behavior of Concrete Overlay Slabs on Unbroken and Broken Base Slabs:

a. Where no initial cracks were present in the base slab or an overlay, the crack patterns produced by traffic overloading were essentially the same for the top and bottom slabs.

b. When top slabs on broken base slabs are overloaded, the original crack pattern of the base slab influences the pattern developed in the top slabs.

c. Where sand-asphalt leveling or cushion courses are used, every precaution should be taken to insure complete curing of the material before placing the concrete top slab.

d. For the two cases tested, no detrimental effect was produced by a lap of 18 inches of a top slab over a free joint in the base pavement.

e. When both overlay and single slab designs were overloaded, the service behavior of the overlay slabs at Lockbourne was better under the action of 37,000 and 60,000 pound wheel load traffic.

SECTION X - RECOMMENDATIONS

10.01 The following recommendations are made as a result of the findings of this investigation:

a. That for wheel loads of 37,000 pounds and greater, the method of design of rigid pavements as given in Chapter XX of the March 1943 edition of the Engineering Manual be revised to give more conservative designs for pavements to be built on plastic clay subgrades.

b. That load transfer devices be used in the design of transverse expansion joints.

c. That pavement units as small as 10 x 10 feet and up to 10" in thickness or less, should not be used in the design of rigid pavements on plastic clay subgrades for wheel loads of 37,000 pounds and greater.

d. That steel reinforcement be included in the design of rigid pavements which are likely to be overloaded.

e. That the tentative Design Criteria for "Portland Cement Overlay Pavements" issued June 1945 by the Office of the Chief of Engineers be considered as producing adequate designs pending further study of this problem.

SECTION XI - BIBLIOGRAPHY

(1) Final Report
Investigation of Concrete Pavements on Different Subgrades
Army Service Forces,
War Department, Corps of Engineers, U. S. Army,
Ohio River Division Laboratories, Mariemont, Ohio
January 1946

(2) Lockbourne Test Track
Design and Construction Report
War Department, Corps of Engineers
Cincinnati Testing Laboratory
Ohio River Division, June 1944

(3) Philippe, R. R.
Structural Behavior of Concrete
Airfield Pavements - The Test Program
Proceedings, Twenty-Fourth Annual Meeting,
Highway Research Board, 1944

(4) Wright Field Slab Tests
Supplementary Report on the Static Loading of
20' x 20' Concrete Slabs
War Department, Corps of Engineers
Ohio River Division, Office of the Division Engineer
Cincinnati Testing Laboratory, Soils Division, January 1943

(5) Report on Special Field Bearing Tests on Natural Subgrade and Prepared Subbase Using Different Size Bearing Plates
War Department, Corps of Engineers
Ohio River Division, Office of the Division Engineer
Cincinnati Testing Laboratory, Soils Division, January 1943

(6) Wright Field Slab Tests, History and Physical Properties of the Concrete
War Department, Corps of Engineers
Cincinnati Testing Laboratory, Concrete Division,
February 1943

(7) Wright Field Slab Tests
Final Report on the Dynamic Loading of Concrete Test Slabs
War Department, Corps of Engineers
Cincinnati Testing Laboratory, Soils Division, August 1943

(8) A Report on Test for the Pavement of the Accelerated Runway Wright Field District
War Department, Corps of Engineers
Soils Laboratory
Wright Field Engineer District, Dayton, Ohio, May 1942

(9) Preliminary Report on the Static Loading of 30 x 12.5 foot Concrete Slabs Cast on a Natural Subgrade
War Department, Corps of Engineers
Wright Field District, May 1942

(10) Report on Service Behavior Tests on Apron Pavement at Godman Field, Fort Knox, Kentucky
U. S. Engineer Office, Louisville, Kentucky, August 1942

(11) Final Report, Service Behavior Tests on Completed Concrete Pavement at Camp Forrest Airfield, Tullahoma, Tennessee
War Department, Corps of Engineers
Cincinnati Testing Laboratory, Soils Section, October 1942

(12) Wright Field Slab Tests
Supplementary Report on the Static Loading of 30 x 12.5 foot Concrete Slabs Cast on Natural Subgrade
War Department, Corps of Engineers
Ohio River Division, Office of the Division Engineer
Cincinnati Testing Laboratory, Soils Division, December 1942

(13) Wright Field Slab Tests
Preliminary Report on the Static Loading of 20 x 30 foot Concrete Slabs Cast on an 18-inch Subbase of Compacted Bank-Run Sand and Gravel
War Department, Corps of Engineers
Office of the Division Engineer
Cincinnati Testing Laboratory, Soils Section, November 1942

(14) Preliminary Report on the Investigation of Concrete Pavements on Different Subgrades
War Department, Corps of Engineers
Wright Field District, December 16, 1941

(15) Westergaard, H. M.
Stresses in Concrete Runways of Airports
Proceedings, Highway Research Board, 1939

(16) Westergaard, H. M., Comdr. CEC, USNR
Technical Memorandum on New Formulas for Stresses and Deflections in Concrete Airfields
Navy Department, Bureau of Yards and Docks, September 1945

(17) Westergaard, H. M.
Stress Concentrations in Plates Loaded Over Small Areas
Transactions of the American Society of Civil Engineers, Vol. 108 (1943) page 831

(18) Westergaard, H. M., Comdr. CEC-V(S) USNR
Progress Report on Critical Stresses at Edges, Joints, and Cracks in Heavy Duty Airfield Pavements Due to Loads, Transmitted Through Tires
To: Chief of Bureau of Yards and Docks, 15 June 1944
NIO-2

(19) Tentative Design Criteria for Portland Cement Overlay Pavements
Army Service Forces, Office of the Chief of Engineers
June 1945

(20) Obert, Leonard and Duvall, W. I.
Discussion of Dynamic Methods of Testing Concrete With Suggestions for Standardization
Proceedings, American Society for Testing Materials, Vol. 41, p. 1053, 1941, Discussion p. 1071

Army Service Forces

Corps of Engineers

LOCKBOURNE NO. 1 - TEST TRACK
FINAL REPORT, ACCELERATED TRAFFIC
TESTS OF CONCRETE PAVEMENTS

TABLES

Ohio River Division Laboratories
Mariemont, Ohio

March 1946

Table I

Results of Maximum Stress Computations

20,000 Pound Wheel Load

Slab Number	Thickness of Concrete in Inches		"k" in lbs./in.^{3}	Maximum stress in lbs./in.^{2}			Avg. Flex. Strength of Concrete lbs./in.^{2}
				Present Method	Alternate Methods		
	Interior	Edge			Interior	Edge	
			6 Inch Slabs				
A2.60	5.75	5.69	108	389	596	898	740
B2.66L	5.50	5.50	110	411	637	938	740
C2.66S	5.50	5.50	72	448	651	1020	740
D2.66	5.50	5.60	72	448	651	994	740
E2.66M	5.75	5.75	108	389	596	885	740
			8 Inch Slabs				
F2.80	7.50	8.00	79	287	386	579	740
G2.8R-0	7.50	7.87	76	289	388	597	740
H2.8R-0	7.12	7.91	69	316	428	602	740
J2.8R-0	7.98	7.98	100	253	339	558	740
			10 Inch Slabs				
K2.100	9.25	9.75	150	192	254	387	740

Table I-A

Design Comparisons

20,000 Pound Wheel Load

Slab Number	Design Factors			Coverages at First Failure	
	Present Method	Alternate Methods		Undoweled End	Doweled End
		Interior	Edge		
		6 Inch Slabs			
A2.60	1.90	1.2	0.82	344 C	440 C
B2.66L	1.80	1.2	0.79	76 L	343 T
C2.66S	1.65	1.1	0.72	38 L	78 L
D2.66	1.65	1.1	0.75	300 T	550 No Failure
E2.66M	1.85	1.2	0.83	202 T	550 No Failure
		8 Inch Slabs			
F2.80	2.58	1.91	1.28	No failure at 550 coverages	
G2.8R-0	2.56	1.90	1.24	" "	"
H2.8R-0	2.34	1.73	1.23	" "	"
J2.8R-0	2.92	2.18	1.32	" "	"
		10 Inch Slabs			
K2.100	3.85	2.90	1.90	" "	"

Table II

Results of Maximum Stress Computations

37,000 Pound Wheel Load

Slab Number	Thickness of Concrete in Inches: Interior	Thickness of Concrete in Inches: Edge	"k" in lbs./in.^{3}	Maximum stress in lbs./in.^{2}: Present Method	Maximum stress in lbs./in.^{2}: Alternate Methods: Interior	Maximum stress in lbs./in.^{2}: Alternate Methods: Edge	Avg. Flex. Strength of Concrete lbs./in.^{2}
				6 Inch Slabs			
AI.60	5.75	5.69	134	494	810	1233	780
BI.66L	5.5	5.5	139	544	845	1339	780
CI.66S	5.5	5.5	88	615	920	1474	780
DI.66	5.5	5.5	76	638	945	1483	780
EI.66M	5.75	5.75	113	546	825	1317	780
RI.612	5.88	5.88	415	362	614	966	780
SI.66	5.83	5.83	393	371	628	989	780
TI.60	5.85	5.41	371	407	702	1108	780
UI.60	5.83	5.83	207	455	720	1158	780
				8 Inch Slabs			
FI.80	7.37	7.37	79	430	605	998	780
MI.86	6.87	7.75	98	451	650	894	780
PI.812	7.79	7.67	84	393	550	931	780
GI.8R-0	7.37	8.00	76	434	609	894	780
HI.8R-0	7.12	7.75	69	464	651	953	780
JI.8R-0	7.44	7.85	100	404	574	873	780
				10 Inch Slabs			
KI.100	9.50	9.25	150	270	371	638	780
OI.106	9.37	9.37	96	300	405	680	780

Table II (Continued)

Results of Maximum Stress Computations

37,000 Pound Wheel Load

Slab Number	Thickness of Concrete in Inches		"k" in lbs./in.^{3}	Maximum stress in lbs./in.^{2}			Avg. Flex. Strength of Concrete lbs./in.^{2}
				Present Method	Alternate Methods		
	Interior	Edge			Interior	Edge	
10 Inch Slabs (Con't)							
QI.1012	9.62	9.62	87	295	354	665	780
Overlay Slabs							
LI.5-60	7.68		92	394	573	901	780
MI.7-60	8.78		91	330	466	946	780
10 Inch Reconstructed Slabs							
AI.106	9.81	9.81	261	231	334	518	725
BI.10R-6	10.06	10.06	207	235	334	525	725
CI.10R-6	10.00	10.00	195	240	340	535	725
8 Inch Reconstructed Slabs							
DI.8R-6	8.18	8.18	157	322	477	739	725
EI.8R-6	8.00	8.00	182	320	481	738	725
FI.8R-6	8.37	8.37	150	315	464	723	725
HI.8R-6	7.94	7.94	88	380	548	866	725

Table II-A

Design Comparisons

37,000 Pound Wheel Load

Slab Number	Design Factor			Coverages at First Failure	
	Present Method	Alternate Methods		Undoweled End	Doweled End
		Interior	Edge		
		6 Inch Slabs			
A1.60	1.58	0.96	0.63	13 L	13 L
B1.66L	1.43	0.92	0.58	14.5 L	14.5 L
C1.66S	1.27	0.85	0.53	15 L	56 L
D1.66	1.22	0.82	0.52	10 L	15 L
E1.66M	1.43	0.94	0.59	16 L	67 L
R1.612	2.18	1.27	0.81	105 C	262 C
S1.6[illegible]	2.10	1.24	0.7[illegible]	65 L	105 L
T1.60	1.91	1.11	0.70	186 L	161 L
U1.60	1.71	1.08	0.67	46 C	130 L
		8 Inch Slabs			
F1.80	1.81	1.29	0.78	111 L	111 L
N1.86	1.73	1.20	0.87	[illegible]07 C	141 C
P1.812	1.98	1.42	0.84	106 T	418 T
G1.8R-O	1.80	1.24	0.87	423 T	300 C
H1.8R-O	1.68	1.11	0.82	--	397 T
J1.8R-O	1.93	1.36	0.89	--	497 L
		10 Inch Slabs			
K1.100	2.80	2.10	1.22	722 T	412 T
O1.106	2.60	1.92	1.15	412 T	2288

Table II-A (Continued)

Design Comparisons

37,000 Pound Wheel Load

Slab Number	Design Factor			Coverages at First Failure	
	Present Method	Alternate Methods		Undoweled End	Doweled End
		Interior	Edge		
		10 Inch Slabs (Con't)			
QI.1012	2.74	1.98	1.17	497 T	2284
		Overlay Slabs			
LI.5-60	1.98	1.36	0.86	272 T	
MI.7-60	2.36	1.67	0.32	423 C	
		10 Inch Reconstructed Slabs			
AI.106	3.12	2.17	1.40	128 C	658 C
BI.10R-6	3.08	2.17	1.38	291 C	521 C
CI.10R-6	3.02	2.13	1.35	1915	1915
		8 Inch Reconstructed Slabs			
DI.8R-6	2.25	1.52	0.98	213 C	253 C
EI.8R-6	2.26	1.50	0.98	421 C	1915
FI.8R-6	2.30	1.56	1.00	248 T	658 C
NI.8R-6	1.90	1.32	0.83	774 T	1915

C = corner break
T = transverse break
L = longitudinal break

Table III

Results of Maximum Stress Computations

60,000 Pound Wheel Load

Slab Number	Thickness of Concrete in Inches		"k" in $lbs./in.^3$	Maximum stress in $lbs./in.^2$			Avg. Flex. Strength of Concrete $lbs./in.^2$
				Present Method	Alternate Methods		
	Interior	Edge			Interior	Edge	
6 Inch Slabs							
R2.612	5.84	5.50	415	491	944	1505	735
S2.66	5.88	5.50	393	481	947	1525	735
T2.60	5.85	5.50	371	495	966	1542	735
U2.60	5.00	5.87	207	706	1357	1588	735
8 Inch Slabs							
G2.8R-0	7.50	7.87	76	603	916	1307	735
H2.8R-0	7.12	7.87	69	661	1007	1323	735
J2.8R-0	7.98	7.98	100	522	798	1215	735
N2.89	8.25	8.00	98	502	603	1215	735
P2.812	7.50	7.37	84	390	902	1406	735
10 Inch Slabs							
K2.100	9.25	9.75	150	392	598	842	735
O2.106	9.55	9.55	96	365	610	949	735
Q2.1012	9.25	9.25	87	442	650	1012	735
Original Overlay Slabs							
L2.5-60	8.24		92	511	760	1156	735
M1.7-60	10.16		91	388	553	865	735

Table III (Continued)

Results of Maximum Stress Computations

60,000 Pound Wheel Load

Slab Number	Thickness of Concrete in Inches		"k" in lbs./in.^{3}	Maximum stress in lbs./in.^{2}			Avg. Flex. Strength of Concrete lbs./in.^{2}
	Interior	Edge		Present Method	Alternate Methods Interior	Alternate Methods Edge	
	Reconstructed Overlay Slabs						
A2.7-60	8.01 *		108	510	772	1160	740
B2.7-66L	7.52 *		110	549	843	1254	740
C2.7-66S	7.52 *		72	609	909	1379	740
D2.7-66	8.47 *		72	520	759	1170	740
E2.7-66M	8.47 *		108	475	710	1076	740
F2.7-80	10.19 *		79	397	563	885	740

* Calculated thickness

Table III-A

Design Comparisons

60,000 Pound Wheel Load

Slab Number	Design Factors			Coverages at First Failure	
	Present Method	Alternate Methods		Undoweled End	Doweled End
		Interior	Edge		
		6 Inch Slab			
R2.612	1.49	0.74	0.49	1.5 C-L	1.5 C
S2.66	1.52	0.78	0.48	1.5 C-L	1.5 C-L
T2.60	1.48	0.76	0.47	1.5 C	42 C-L
U2.60	1.04	0.54	0.46	1.5 L	1.5 L
		8 Inch Slab			
G2.8R-0	1.22	0.80	0.56	1.5 T	32 T
H2.8R-0	1.11	0.73	0.55	--	42 T
J2.8R-0	1.41	0.92	0.60	--	138 C
N2.86	1.46	1.22	0.60	1.5 C	16 C
P2.812	1.25	0.81	0.52	1.5 T	3 T
		10 Inch Slab			
K2.100	1.88	1.23	0.87	138 T	6 T
O2.106	2.01	1.20	0.78	72 C	72 T
Q2.1012	1.66	1.13	0.73	42 C	72 C
		Overlay Slabs			
L2.5-60	1.43	0.97	0.64	32 C	--
M1.7-60	1.94	1.33	0.85	98 C	--

Table III-A (Continued)

Design Comparisons

60,000 Pound Wheel Load

Slab Number	Design Factors			Coverages at First Failure	
	Present Method	Alternate Methods		Undoweled End	Doweled End
		Interior	Edge		
	Reconstructed Overlay Slabs				
A2.7-60	1.44	0.95	0.63	52 C	32 C
B2.7-66L	1.34	0.87	0.58	24 C	16 L
C2.7-66S	1.21	0.81	0.53	1.5 C	6 L
D2.7-66	1.41	0.97	0.63	98 C	42 C
E2.7-66M	1.55	1.03	0.68	16 C	98 T
F2.7-80	1.85	1.30	0.83	42 C	98 C-T

C = corner break
T = transverse break
L = longitudinal break

Army Service Forces

Corps of Engineers

LOCKBOURNE NO. 1 - TEST TRACK
FINAL REPORT, ACCELERATED TRAFFIC
TESTS OF CONCRETE PAVEMENTS

FIGURES

Ohio River Division Laboratories
Mariemont, Ohio

March 1946

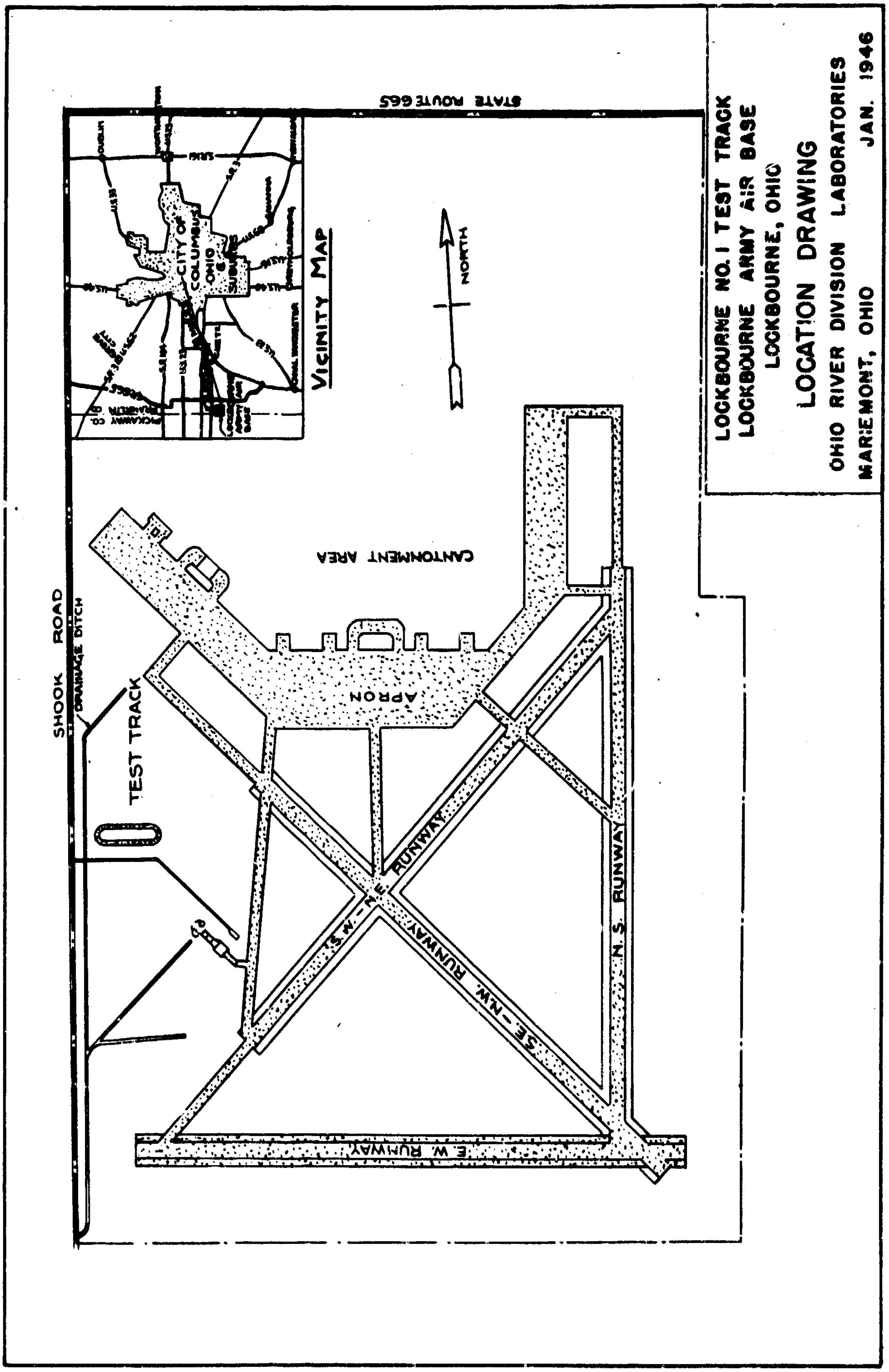

LOCKBOURNE NO. 1 TEST TRACK
LOCKBOURNE ARMY AIR BASE
LOCKBOURNE, OHIO
LOCATION DRAWING
OHIO RIVER DIVISION LABORATORIES
MARIEMONT, OHIO
JAN. 1946
VICINITY MAP
CITY OF COLUMBUS OHIO & SUBURBS
PICKAWAY CO.
NORTH
STATE ROUTE 665
CANTONMENT AREA
APRON
SHOOK ROAD
DRAINAGE DITCH
TEST TRACK
SE.-NW. RUNWAY
S.W.-N.E. RUNWAY
N.S. RUNWAY
E.W. RUNWAY

FIGURE 1.0

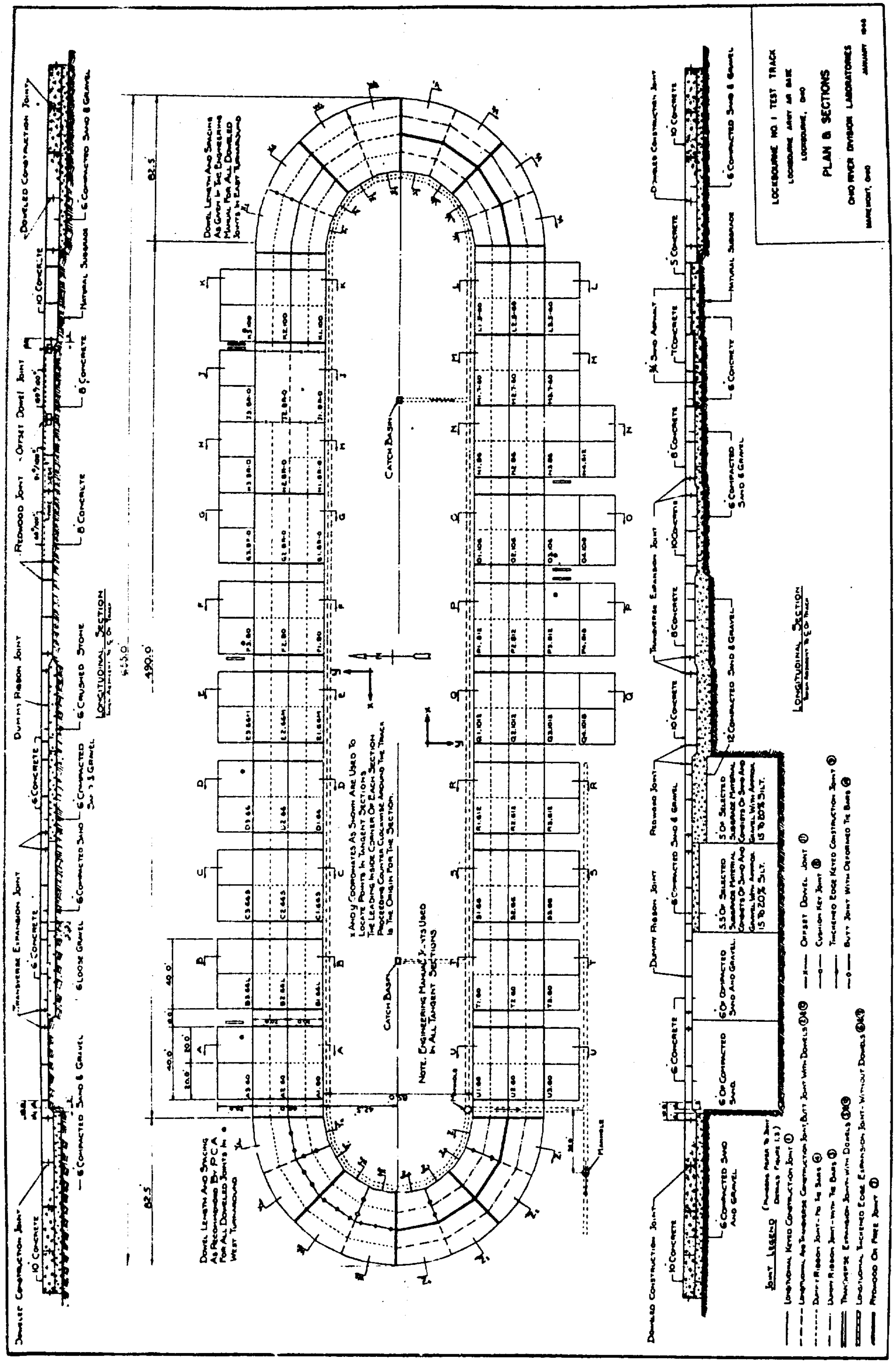

LOCKBOURNE NO. 1 TEST TRACK
PLAN & SECTIONS
OHIO RIVER DIVISION LABORATORIES

FIGURE 1.1

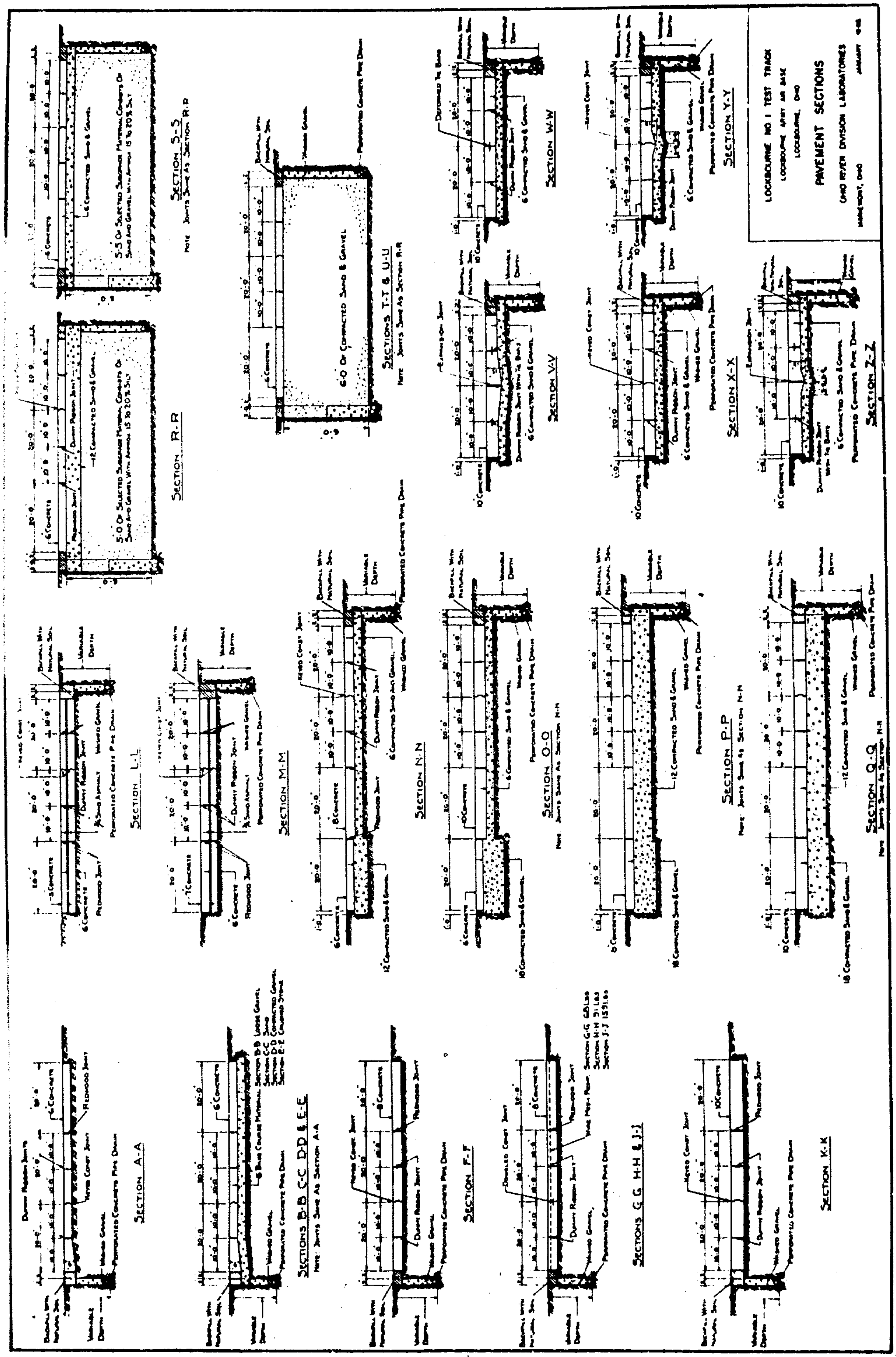


FIGURE 1.2

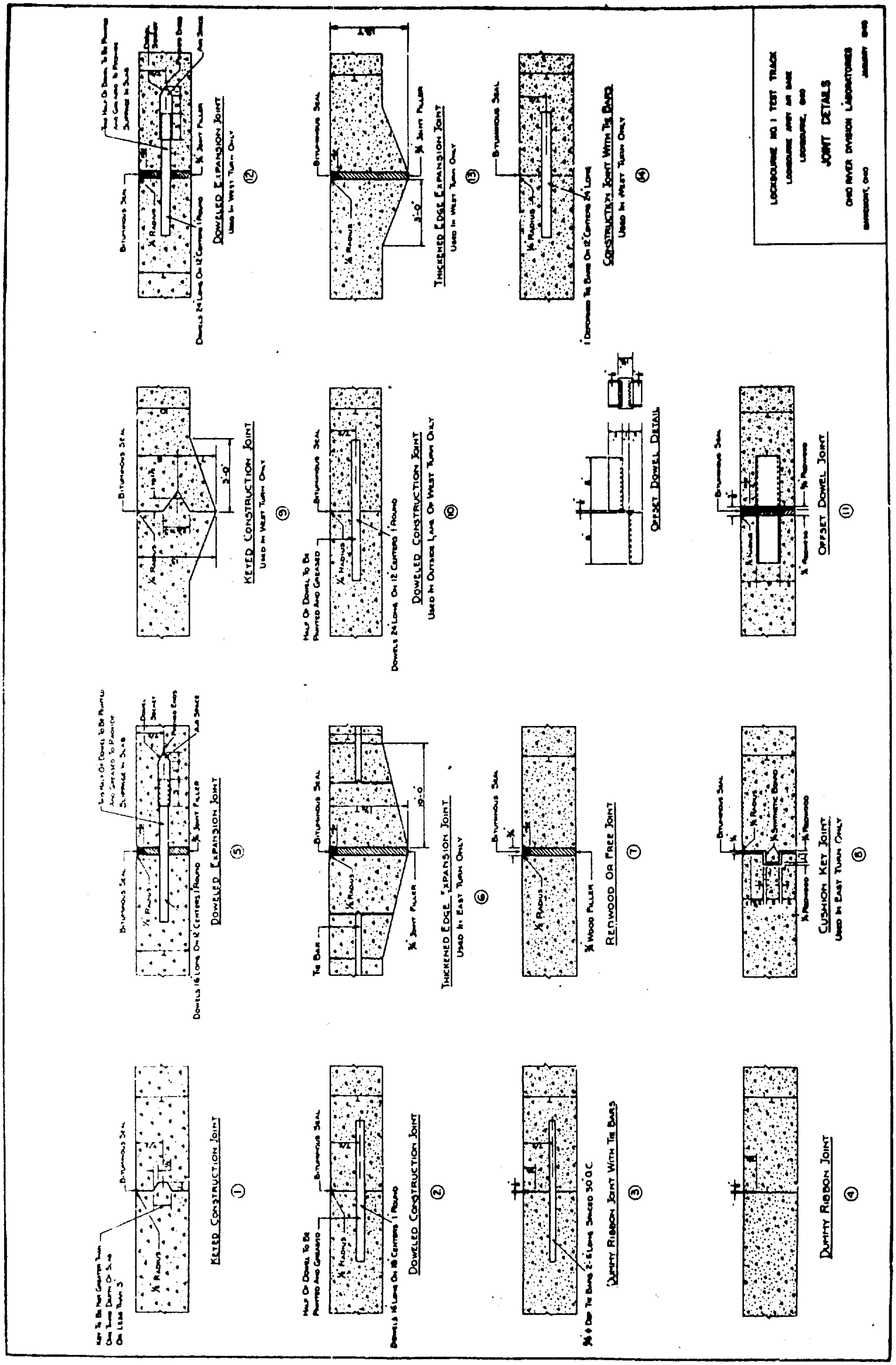

Lockbourne No. 1 Test Track
Joint Details
Ohio River Division Laboratories
Keyed Construction Joint
Doweled Construction Joint
Dummy Ribbon Joint With Tie Bars
Dummy Ribbon Joint
Doweled Expansion Joint
Thickened Edge Expansion Joint
Used In East Turn Only
Renwood Or Free Joint
Cushion Key Joint
Used In East Turn Only
Keyed Construction Joint
Used In West Turn Only
Doweled Construction Joint
Used In Outside Lane Of West Turn Only
Offset Dowel Detail
Offset Dowel Joint
Doweled Expansion Joint
Used In West Turn Only
Thickened Edge Expansion Joint
Used In West Turn Only
Construction Joint With Tie Bars
Used In West Turn Only
Bituminous Seal

FIGURE 1.3

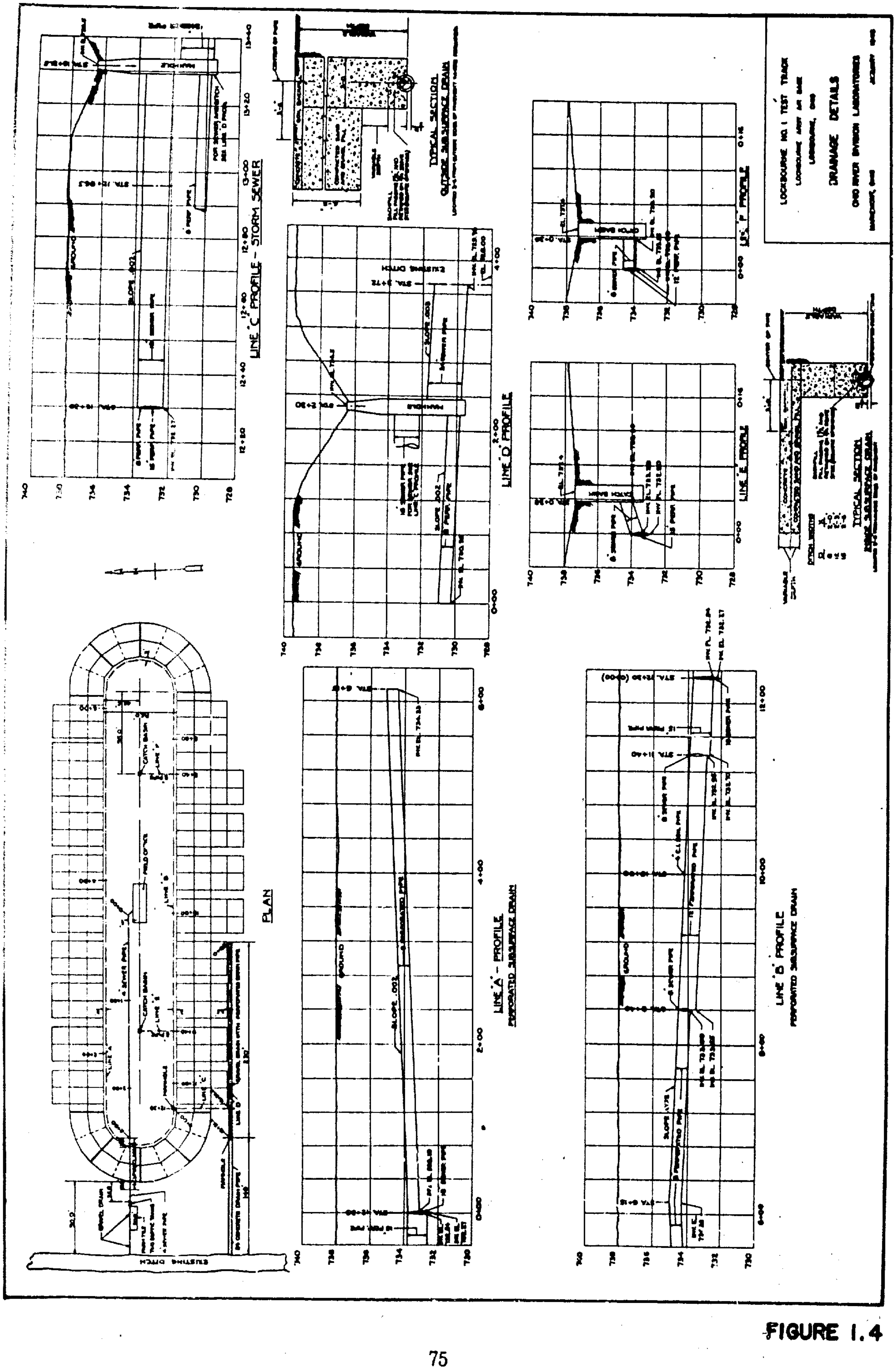

LOCKBOURNE NO. 1 TEST TRACK
DRAINAGE DETAILS
PLAN
LINE 'A' - PROFILE
PERFORATED SUBSURFACE DRAIN
LINE 'B' PROFILE
PERFORATED SUBSURFACE DRAIN
LINE 'C' PROFILE - STORM SEWER
LINE 'D' PROFILE
LINE 'E' PROFILE
LINE 'F' PROFILE
TYPICAL SECTION
OUTSIDE SUBSURFACE DRAIN
TYPICAL SECTION
INSIDE SUBSURFACE DRAIN
EXISTING DITCH
MANHOLE
CATCH BASIN
FIELD OFFICE

FIGURE 1.4

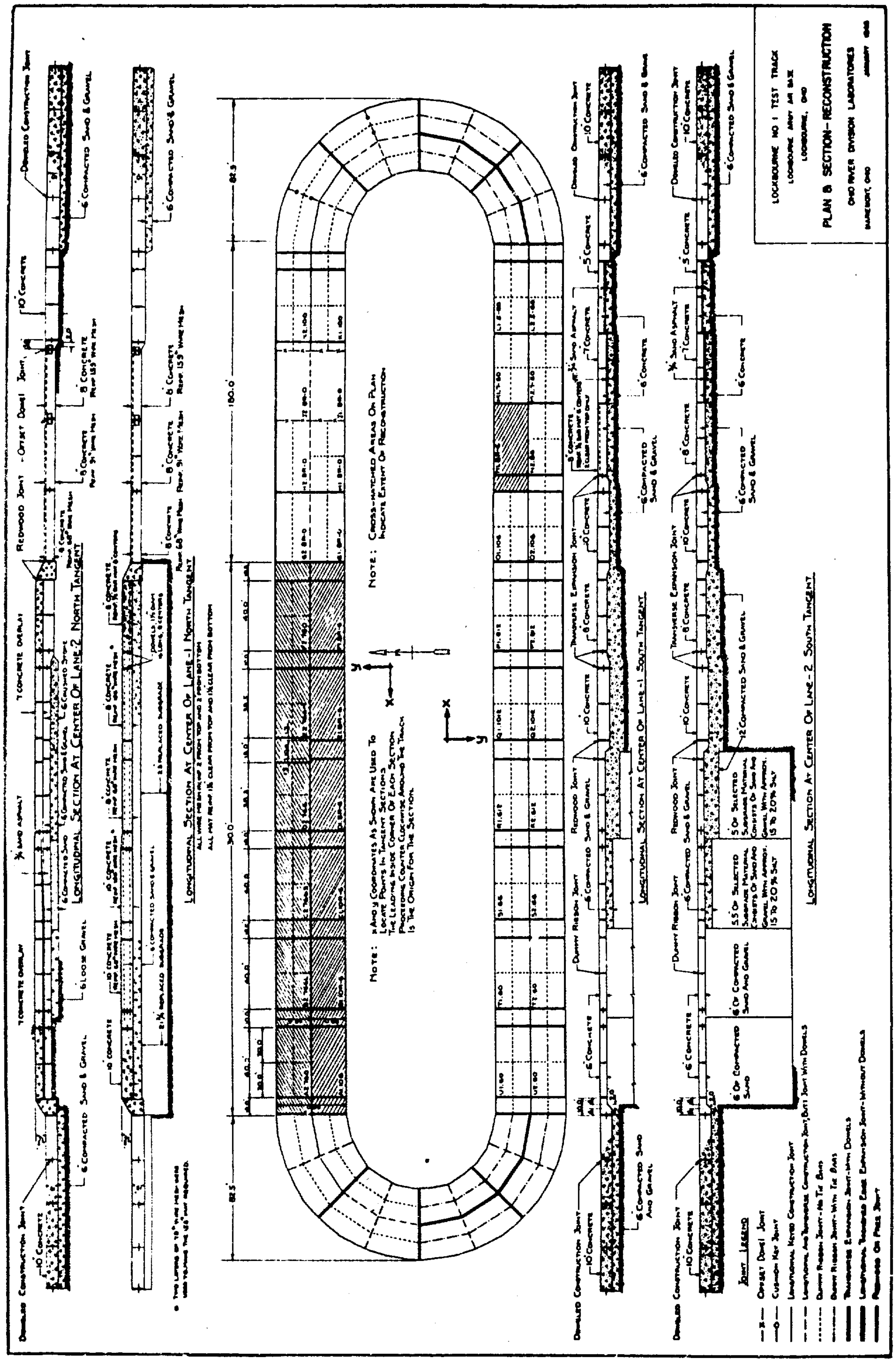
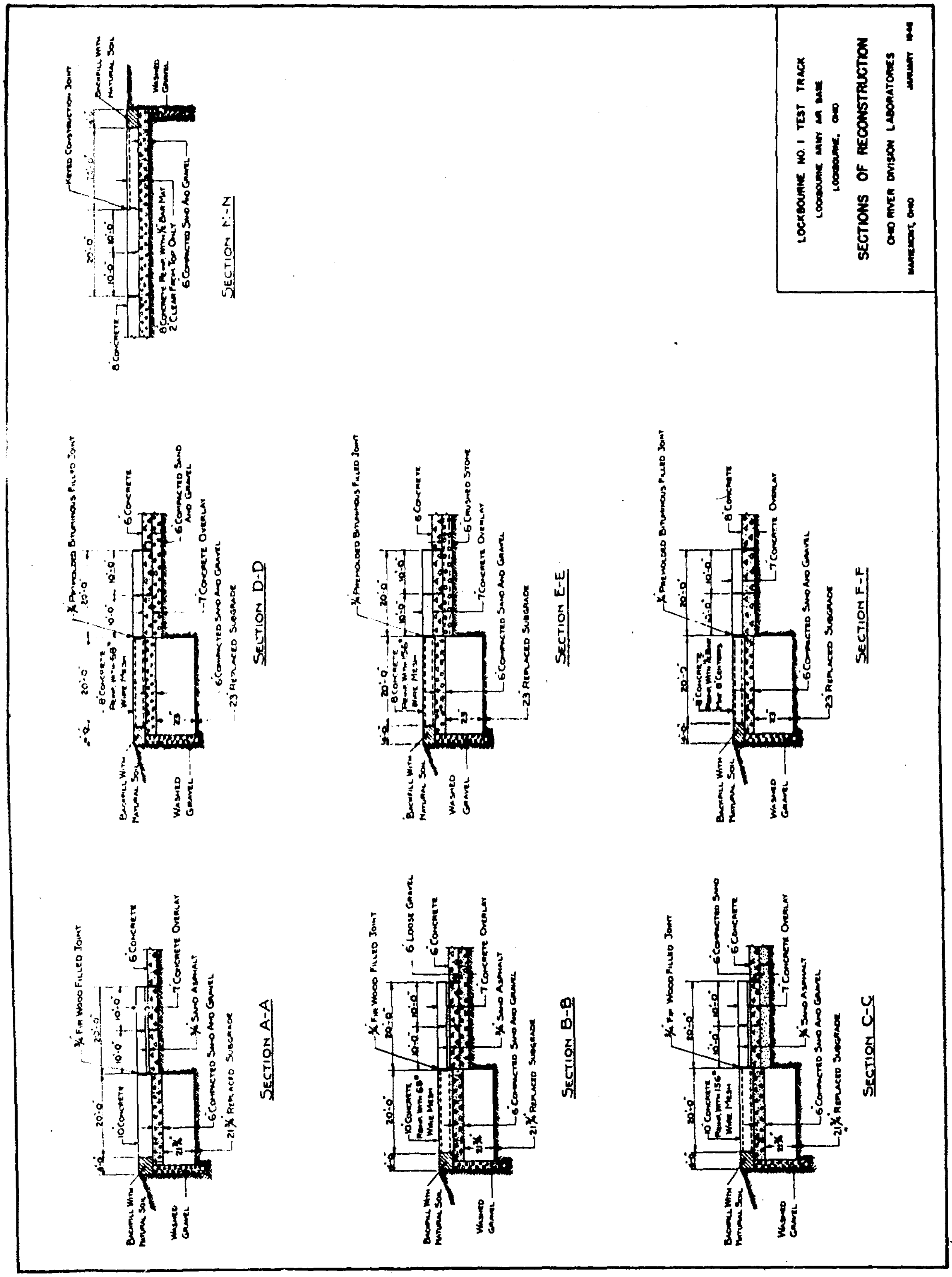

LOCKBOURNE NO 1 TEST TRACK
PLAN & SECTION—RECONSTRUCTION
OHIO RIVER DIVISION LABORATORIES

FIGURE 1.5

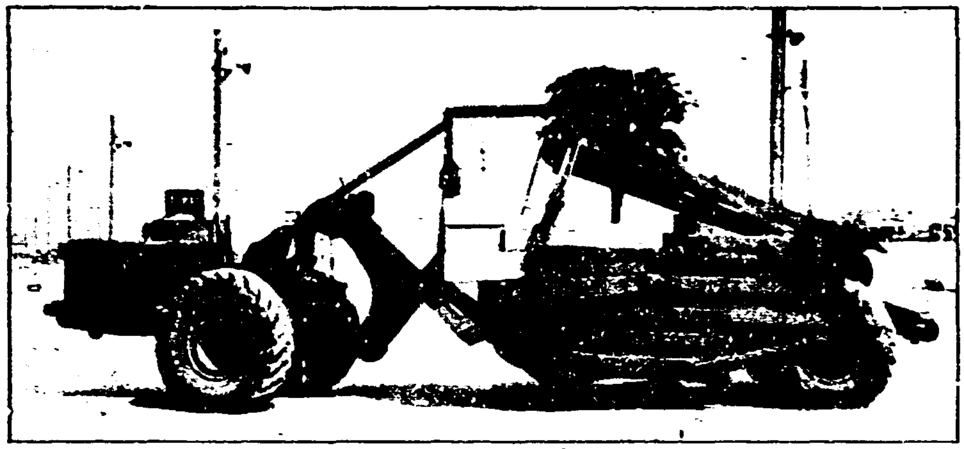


LOCKBOURNE NO. 1 TEST TRACK
LOCKBOURNE ARMY AIR BASE
LOCKBOURNE, OHIO
SECTIONS OF RECONSTRUCTION
OHIO RIVER DIVISION LABORATORIES
MARIEMONT, OHIO
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F

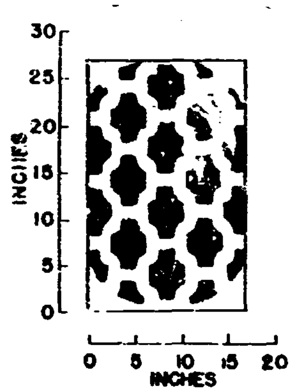
FIGURE 1.6

FINAL REPORT LOCKBOURNE NO. 1

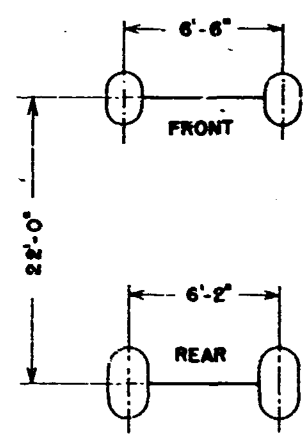
EQUIPMENT USED FOR 20,000 POUND WHEEL LOAD TRAFFIC



SUPER C TOURNAPULL WITH MODEL LP SCRAPER LOADED WITH 1 TON CONCRETE BLOCKS
SPEED OF TRAFFIC FROM 5 TO 10 MILES PER HOUR



TYPICAL TIRE PRINT
INFLATION PRESSURE 40 LBS. PER SQUARE INCH
AREA 387 SQUARE INCHES



SPACING OF WHEELS

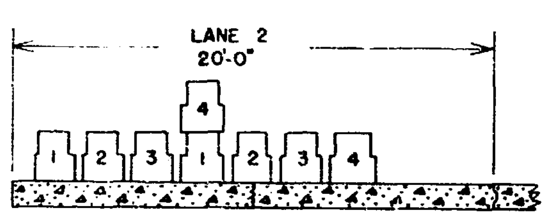
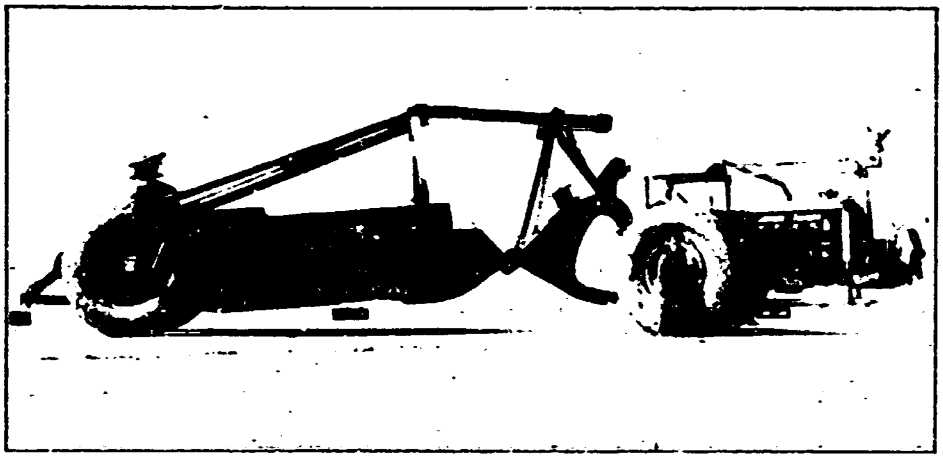


DIAGRAM OF TRAFFIC DISTRIBUTION
THE 4 TRIPS INDICATED ON THE DIAGRAM ARE ASSUMED TO MAKE 2 COVERAGES

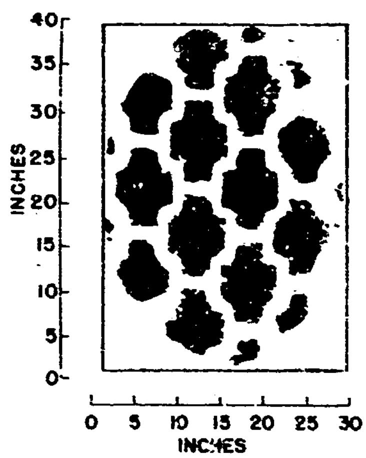
FIGURE 2.0

FINAL REPORT LOCKBOURNE NO. 1

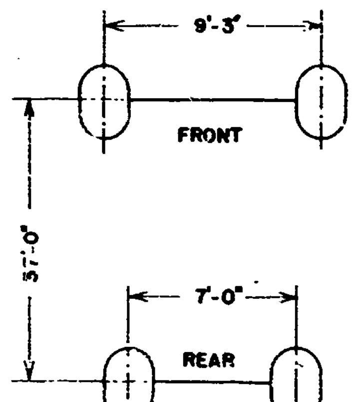
EQUIPMENT USED FOR 37,000 POUND WHEEL LOAD TRAFFIC



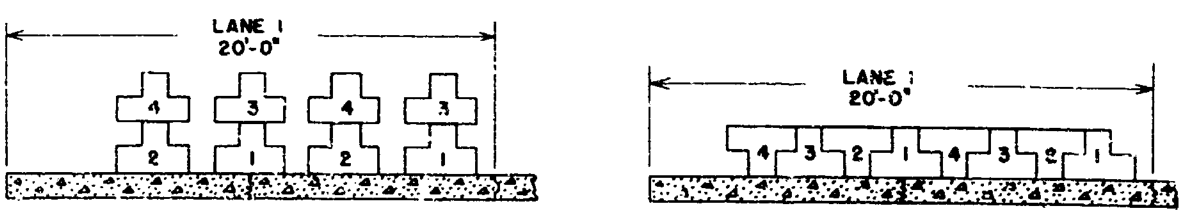
A-3 TOURNAPULL WITH MODEL NU SCRAPER LOADED WITH 1 TON CONCRETE BLOCKS
SPEED OF TRAFFIC FROM 4 TO 5 MILES PER HOUR



TYPICAL TIRE PRINT
INFLATION PRESSURE 47 LBS. PER SQUARE INCH
AREA 638 SQUARE INCHES



SPACING OF WHEELS



0-377 COVERAGES

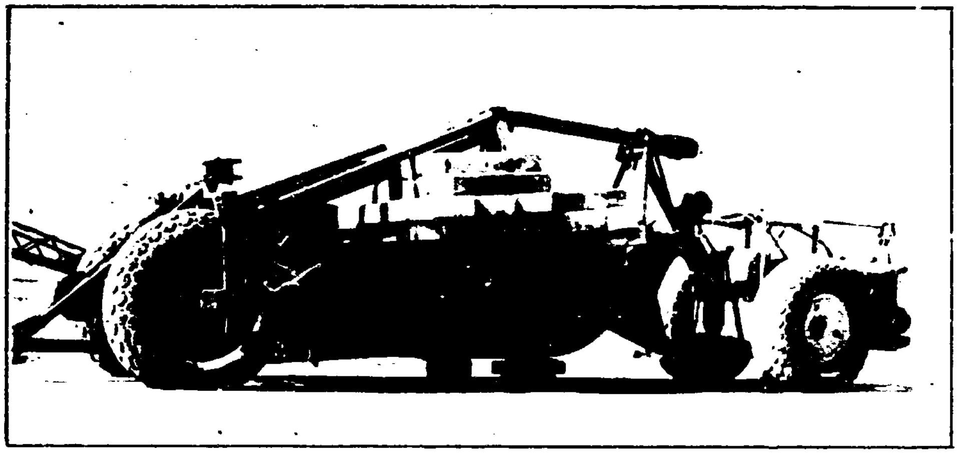
377-2285 COVERAGES

DIAGRAMS OF TRAFFIC DISTRIBUTION
THE 4 TRIPS INDICATED ON EACH DIAGRAM
ARE ASSUMED TO MAKE 2 COVERAGES

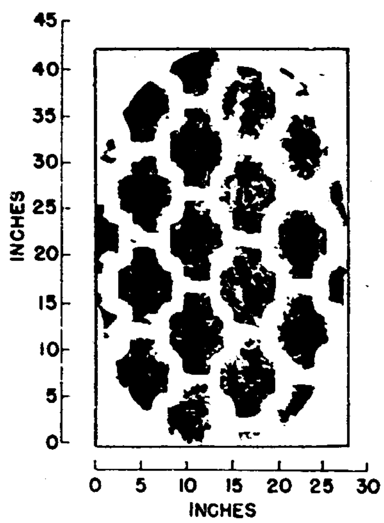
FIGURE 2.1

FINAL REPORT LOCKBOURNE NO. I

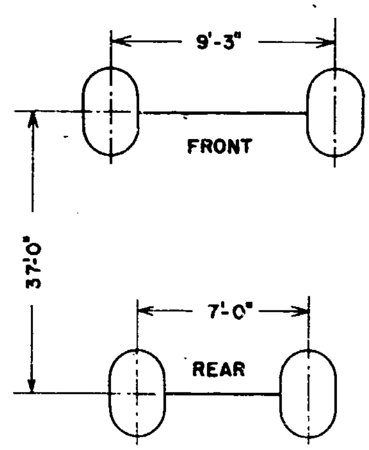
EQUIPMENT USED FOR 60,000 POUND WHEEL LOAD TRAFFIC



A-3 TOURNAPULL WITH MODEL NU SCRAPER LOADED WITH I TON CONCRETE BLOCKS
SPEED OF TRAFFIC FROM 3.0 TO 3.5 MILES PER HOUR



TYPICAL TIRE PRINT
INFLATION PRESSURE 52 LBS. PER SQUARE INCH
AREA 1051 SQUARE INCHES



SPACING OF WHEELS

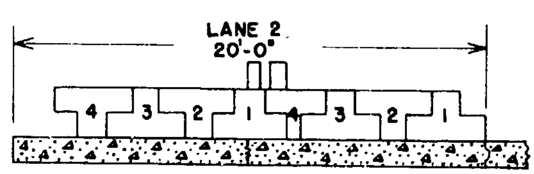
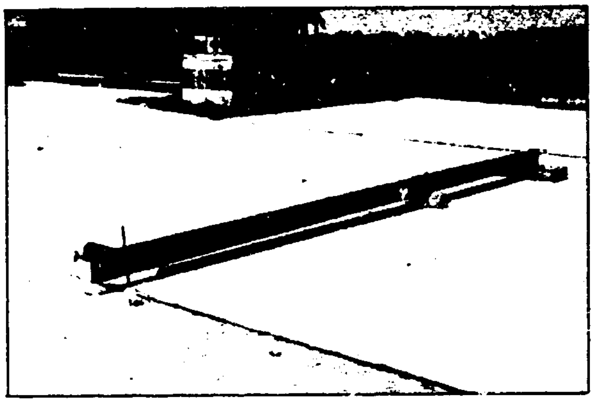


DIAGRAM OF TRAFFIC DISTRIBUTION
THE 4 TRIPS INDICATED ON THE DIAGRAM
ARE ASSUMED TO MAKE 2 COVERAGES

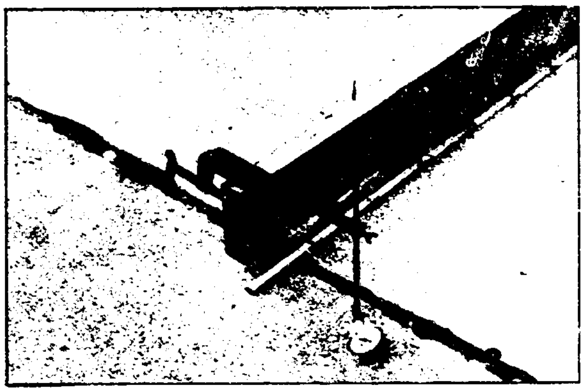
FIGURE 2.2

FINAL REPORT LOCKBOURNE NO. 1

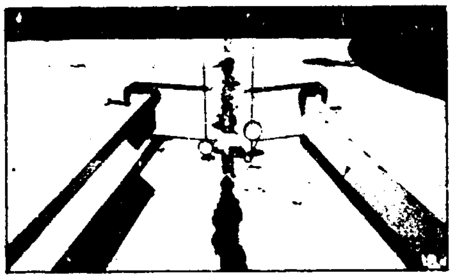
ARRANGEMENT OF EQUIPMENT FOR MEASUREMENT OF PAVEMENT DEFLECTIONS UNDER STATIONARY WHEEL LOADS



(A) WOODEN CANTILEVER AND EXTENSOMETER.



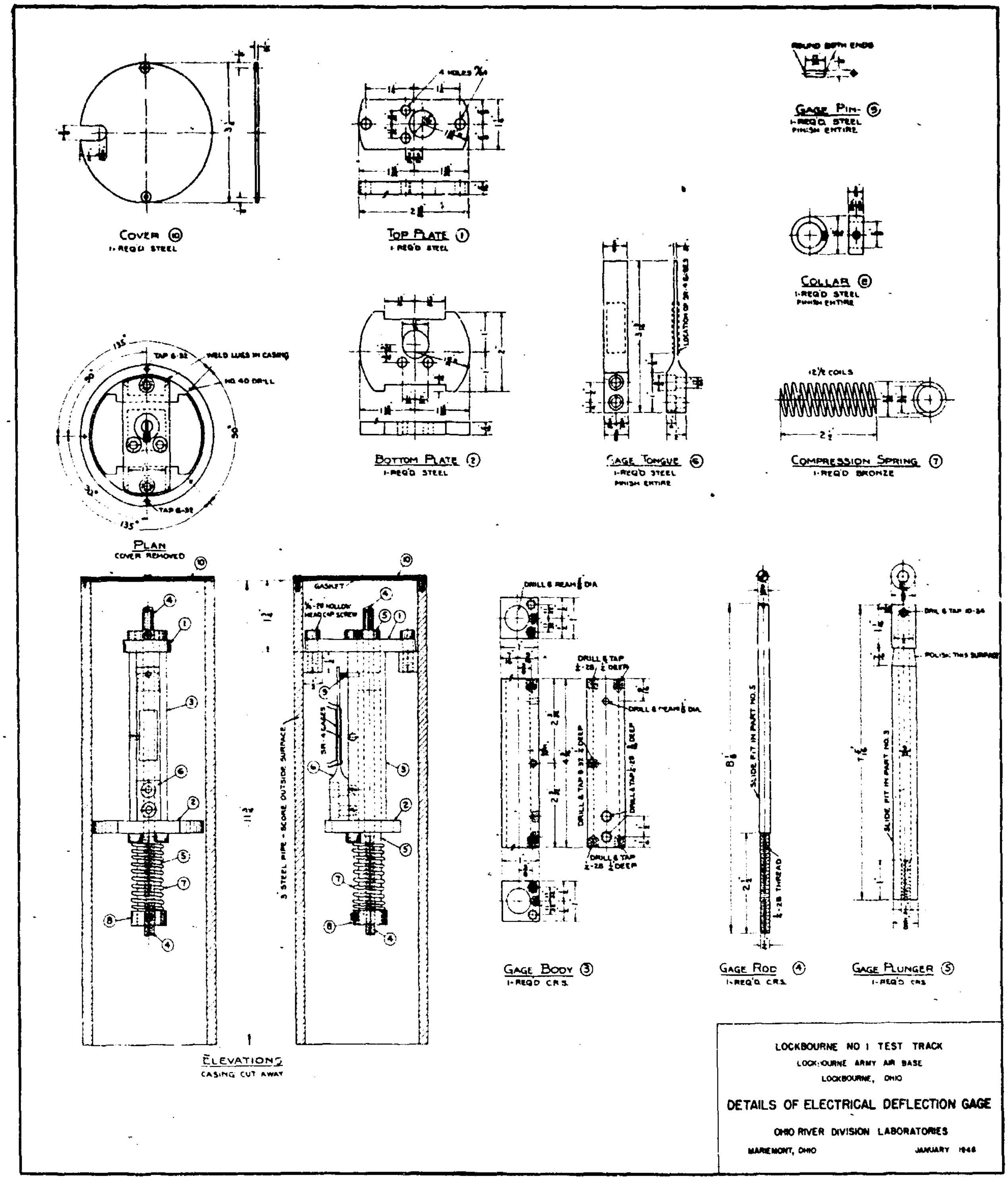
(B) DETAIL OF EXTENSOMETER AND SUPPORT.



(C) ARRANGEMENT FOR DEFLECTION MEASUREMENT AT A JOINT.

(D) MEASUREMENT OF DEFLECTION FOR STATIONARY WHEEL LOAD.

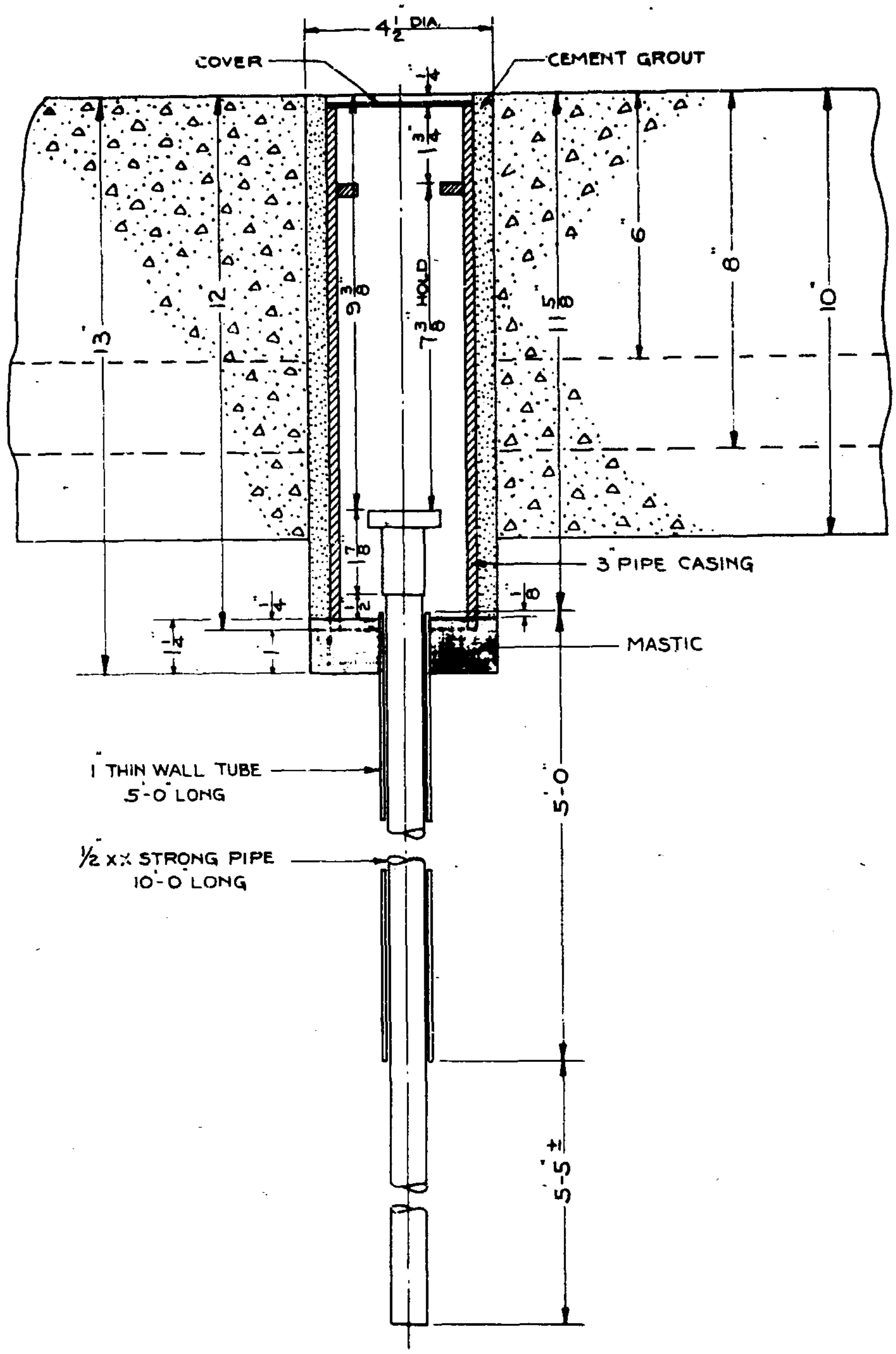
FIGURE 2.3



COVER ⑩
1-REQ'D STEEL
TOP PLATE ①
1-REQ'D STEEL
GAGE PIN ⑤
1-REQ'D STEEL
FINISH ENTIRE
COLLAR ⑧
1-REQ'D STEEL
FINISH ENTIRE
BOTTOM PLATE ②
1-REQ'D STEEL
GAGE TONGUE ⑥
1-REQ'D STEEL
FINISH ENTIRE
COMPRESSION SPRING ⑦
1-REQ'D BRONZE
PLAN
COVER REMOVED
ELEVATIONS
CASING CUT AWAY
GAGE BODY ③
1-REQ'D C.R.S.
GAGE ROD ④
1-REQ'D C.R.S.
GAGE PLUNGER ⑤
1-REQ'D C.R.S.
LOCKBOURNE NO 1 TEST TRACK
LOCKBOURNE ARMY AIR BASE
LOCKBOURNE, OHIO
DETAILS OF ELECTRICAL DEFLECTION GAGE
OHIO RIVER DIVISION LABORATORIES
MARIEMONT, OHIO
JANUARY 1946

FIGURE 2.4

LOCKBOURNE NO. 1 TEST TRACK



DETAILS OF INSTALLATION OF METAL CASING AND REFERANCE HUB
FOR WOODMAN ELECTRICAL DEFLECTION GAGES

FIGURE 2.5

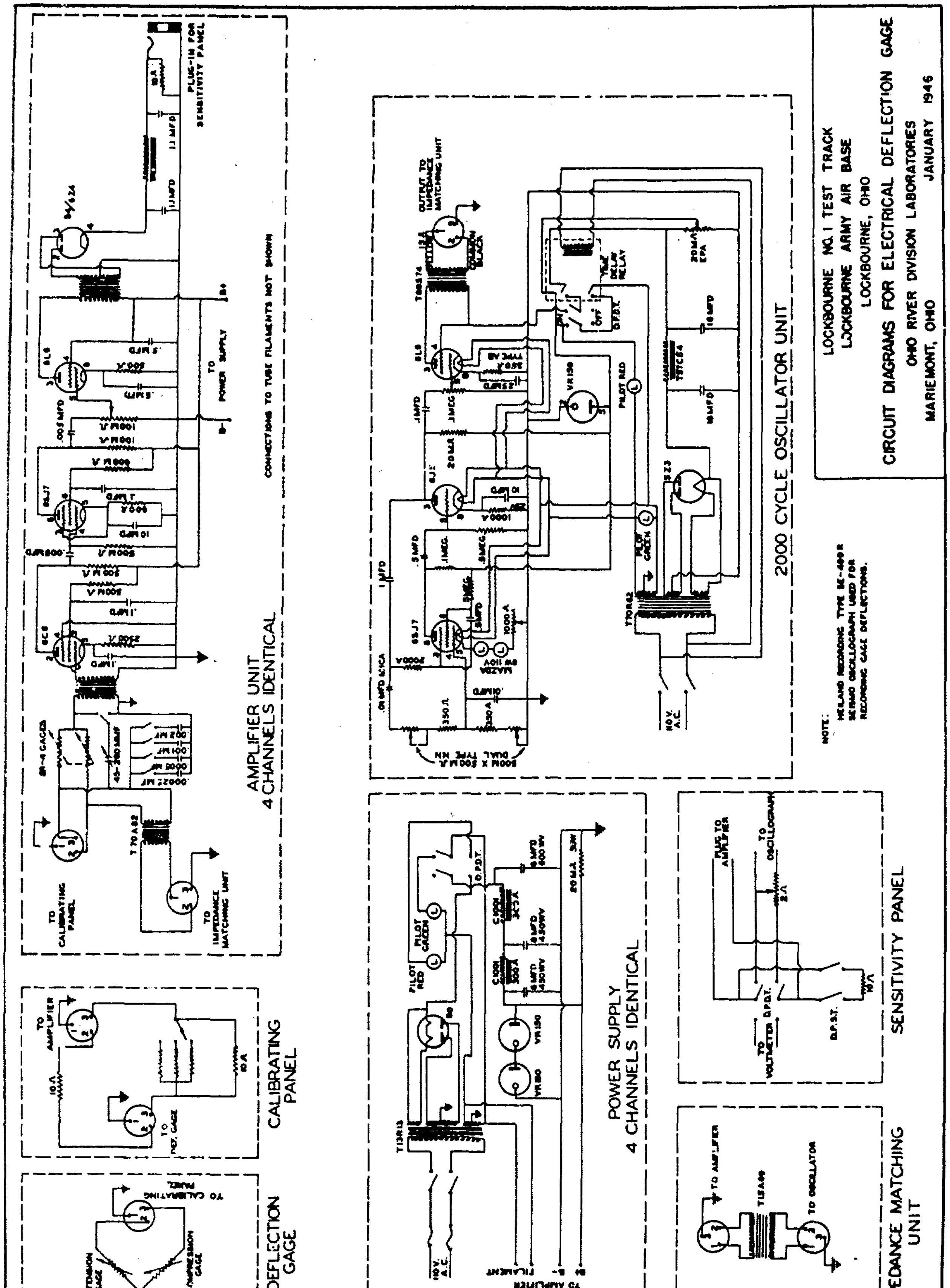

LOCKBOURNE NO. 1 TEST TRACK
LOCKBOURNE ARMY AIR BASE
LOCKBOURNE, OHIO
CIRCUIT DIAGRAMS FOR ELECTRICAL DEFLECTION GAGE
OHIO RIVER DIVISION LABORATORIES
MARIEMONT, OHIO
JANUARY 1946
AMPLIFIER UNIT
4 CHANNELS IDENTICAL
CONNECTIONS TO TUBE FILAMENTS NOT SHOWN
2000 CYCLE OSCILLATOR UNIT
POWER SUPPLY
4 CHANNELS IDENTICAL
SENSITIVITY PANEL
IMPEDANCE MATCHING
UNIT
CALIBRATING
PANEL
DEFLECTION
GAGE

FIGURE 2.6

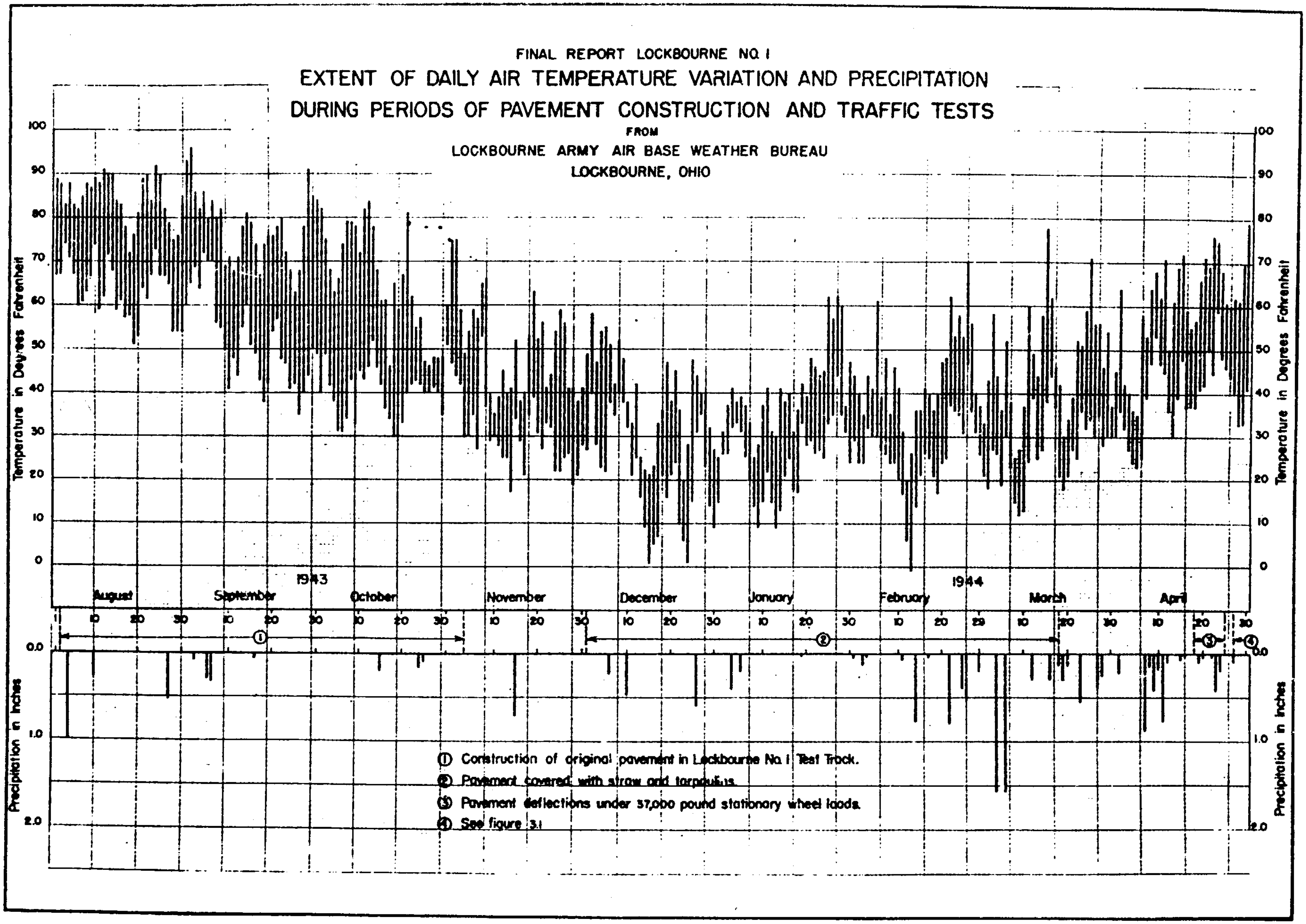

FINAL REPORT LOCKBOURNE NO. I
EXTENT OF DAILY AIR TEMPERATURE VARIATION AND PRECIPITATION
DURING PERIODS OF PAVEMENT CONSTRUCTION AND TRAFFIC TESTS
FROM
LOCKBOURNE ARMY AIR BASE WEATHER BUREAU
LOCKBOURNE, OHIO
Temperature in Degrees Fahrenheit
Precipitation in Inches
1943
1944
August
September
October
November
December
January
February
March
April
① Construction of original pavement in Lockbourne No. I Test Track.
② Pavement covered with straw and tarpaulins.
③ Pavement deflections under 37,000 pound stationary wheel loads.
④ See figure 3.1

FIGURE 3.0

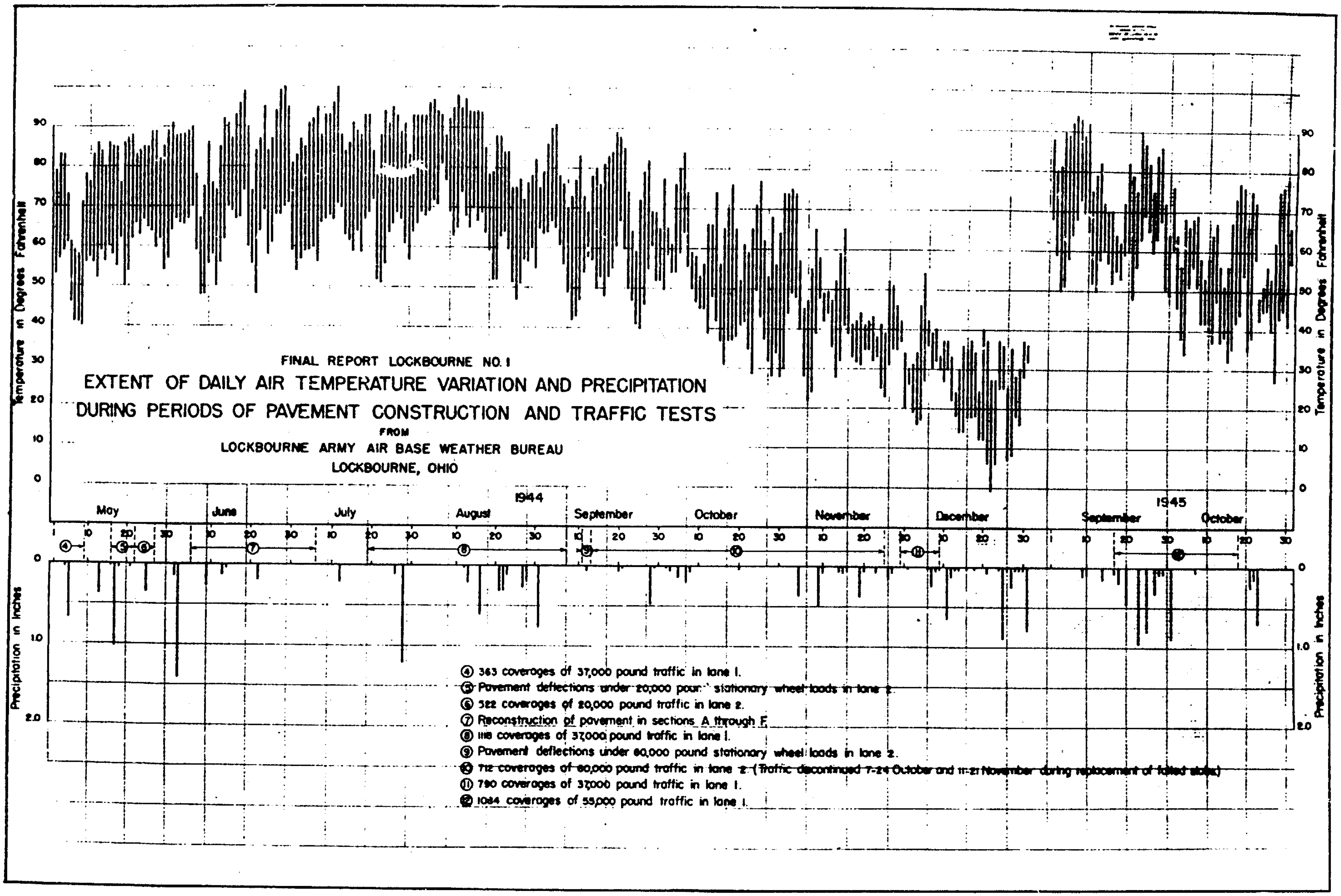

FINAL REPORT LOCKBOURNE NO. 1
EXTENT OF DAILY AIR TEMPERATURE VARIATION AND PRECIPITATION
DURING PERIODS OF PAVEMENT CONSTRUCTION AND TRAFFIC TESTS
FROM
LOCKBOURNE ARMY AIR BASE WEATHER BUREAU
LOCKBOURNE, OHIO
Temperature in Degrees Fahrenheit
Precipitation in Inches
1944
1945
May
June
July
August
September
October
November
December
④ 363 coverages of 37,000 pound traffic in lane 1.
⑤ Pavement deflections under 20,000 pound stationary wheel loads in lane 2.
⑥ 522 coverages of 20,000 pound traffic in lane 2.
⑦ Reconstruction of pavement in sections A through F.
⑧ 118 coverages of 37,000 pound traffic in lane 1.
⑨ Pavement deflections under 60,000 pound stationary wheel loads in lane 2.
⑩ 712 coverages of 60,000 pound traffic in lane 2. (Traffic discontinued 7-24 October and 11-21 November during replacement of failed slabs)
⑪ 790 coverages of 37,000 pound traffic in lane 1.
⑫ 1084 coverages of 55,000 pound traffic in lane 1.

FIGURE 3.1

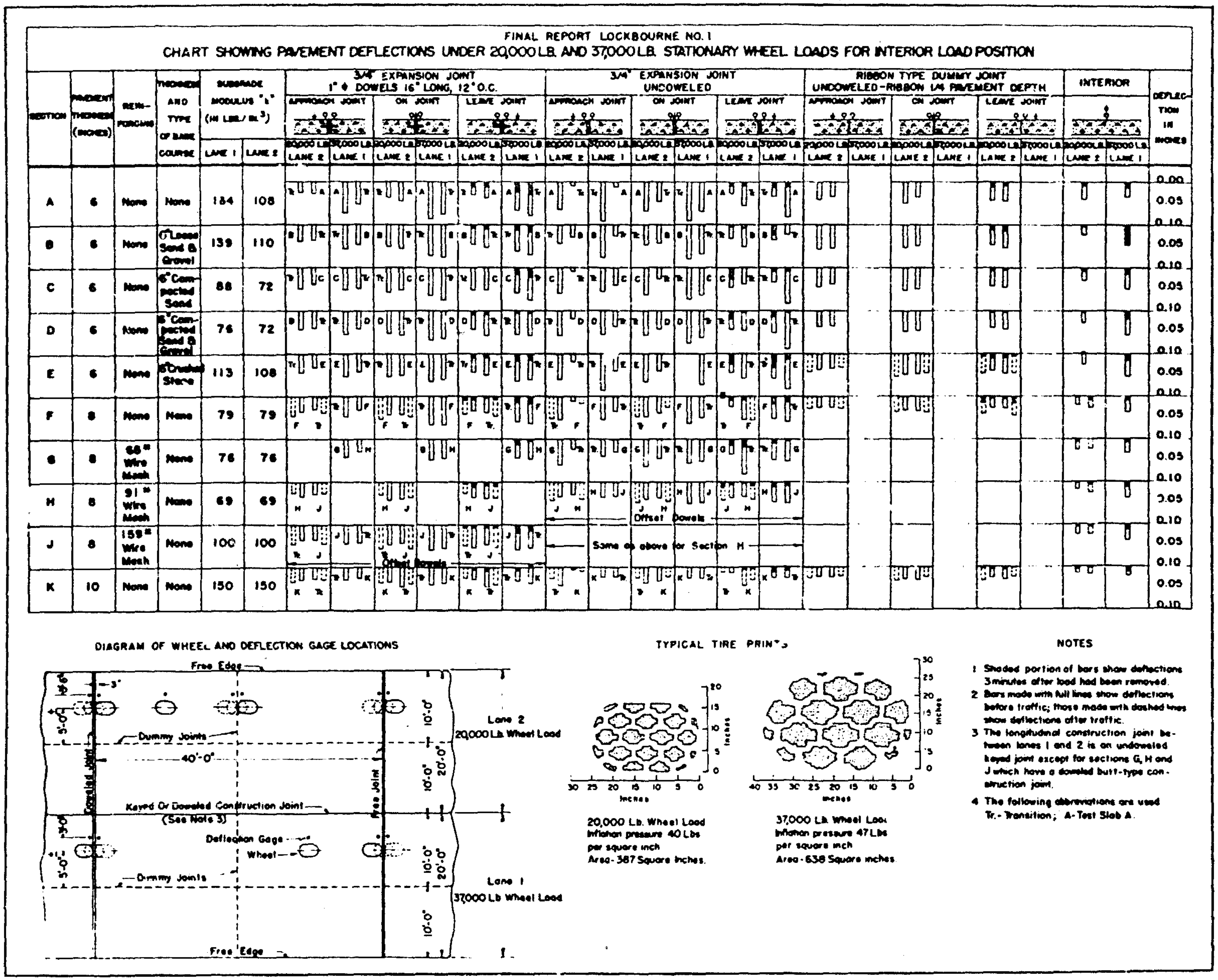

FINAL REPORT LOCKBOURNE NO. 1
CHART SHOWING PAVEMENT DEFLECTIONS UNDER 20,000 LB. AND 37,000 LB. STATIONARY WHEEL LOADS FOR INTERIOR LOAD POSITION
SECTION
PAVEMENT THICKNESS (INCHES)
REIN-FORCING
THICKNESS AND TYPE OF BASE COURSE
SUBGRADE MODULUS "k" (IN LBS./IN.³)
LANE 1
LANE 2
3/4" EXPANSION JOINT 1" φ DOWELS 16" LONG, 12" O.C.
3/4" EXPANSION JOINT UNDOWELED
RIBBON TYPE DUMMY JOINT UNDOWELED-RIBBON 1/4 PAVEMENT DEPTH
INTERIOR
APPROACH JOINT
ON JOINT
LEAVE JOINT
20,000 LB.
37,000 LB.
DEFLECTION IN INCHES
A 6 None None 184 108
B 6 None 6" Loose Sand & Gravel 139 110
C 6 None 6" Compacted Sand 88 72
D 6 None 6" Compacted Sand & Gravel 76 72
E 6 None 6" Crushed Stone 113 108
F 8 None None 79 79
G 8 68" Wire Mesh None 76 76
H 8 91" Wire Mesh None 69 69
J 8 159" Wire Mesh None 100 100
K 10 None None 150 150
Offset Dowels
Same as above for Section H
0.00
0.05
0.10
DIAGRAM OF WHEEL AND DEFLECTION GAGE LOCATIONS
Free Edge
Dummy Joints
40'-0"
Keyed Or Doweled Construction Joint (See Note 3)
Deflection Gage
Wheel
Free Joint
Doweled Joint
10'-0"
20'-0"
5'-0"
Lane 2
20,000 Lb. Wheel Load
Lane 1
37,000 Lb. Wheel Load
TYPICAL TIRE PRINTS
Inches
20,000 Lb. Wheel Load
Inflation pressure 40 Lbs per square inch
Area - 387 Square Inches.
37,000 Lb. Wheel Load
Inflation pressure 47 Lbs per square inch
Area - 638 Square inches
NOTES
1 Shaded portion of bars show deflections 3 minutes after load had been removed.
2 Bars made with full lines show deflections before traffic; those made with dashed lines show deflections after traffic.
3 The longitudinal construction joint between lanes 1 and 2 is on undoweled keyed joint except for sections G, H and J which have a doweled butt-type construction joint.
4 The following abbreviations are used Tr. - Transition; A - Test Slab A.

FIGURE 4.0

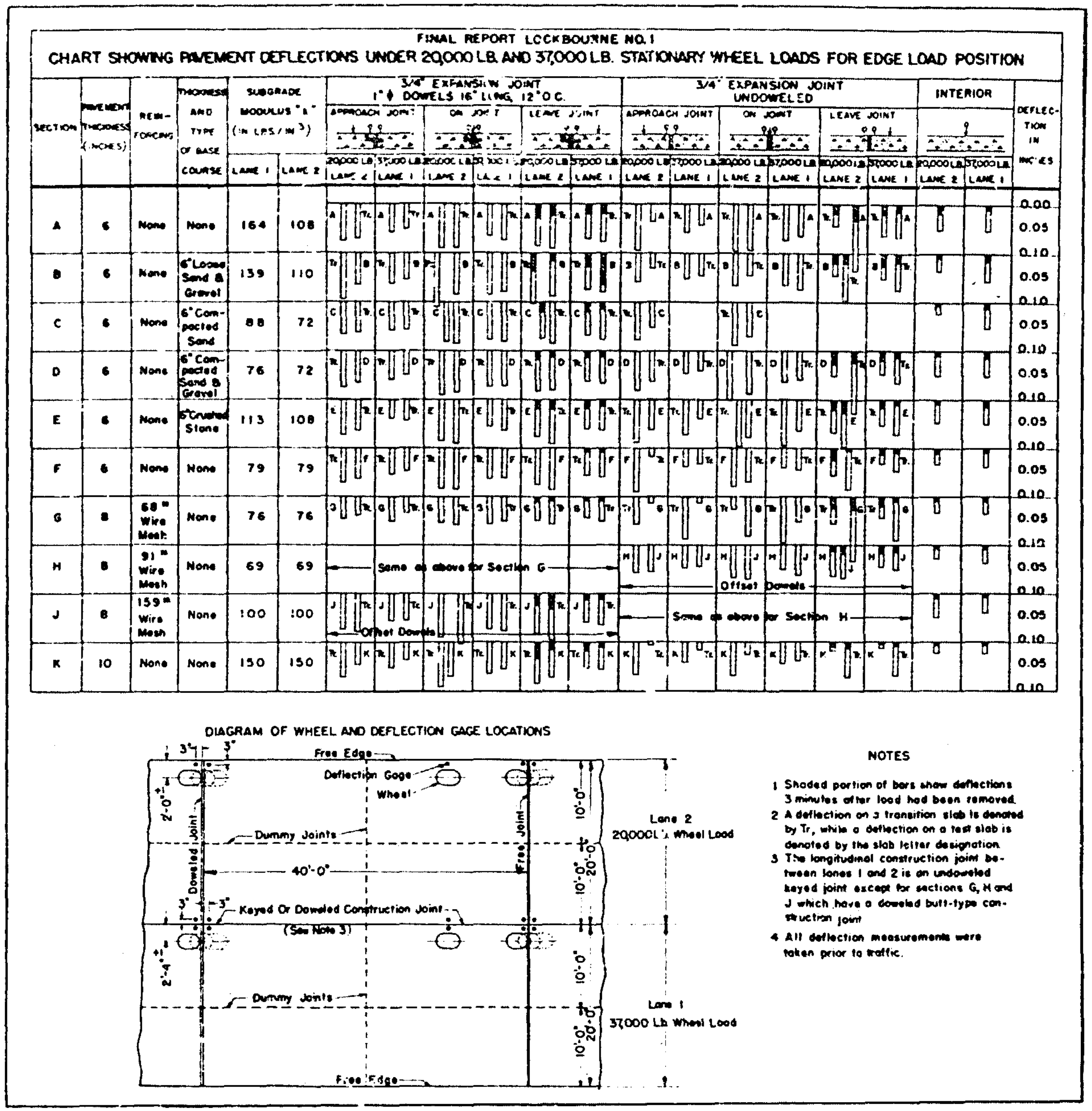

FINAL REPORT LOCKBOURNE NO. 1
CHART SHOWING PAVEMENT DEFLECTIONS UNDER 20,000 LB. AND 37,000 LB. STATIONARY WHEEL LOADS FOR EDGE LOAD POSITION
SECTION
PAVEMENT THICKNESS (INCHES)
REIN-FORCING
THICKNESS AND TYPE OF BASE COURSE
SUBGRADE MODULUS "k" (IN LBS/IN³)
LANE 1
LANE 2
3/4" EXPANSION JOINT 1" φ DOWELS 16" LONG, 12" O.C.
3/4" EXPANSION JOINT UNDOWELED
INTERIOR
APPROACH JOINT
ON JOINT
LEAVE JOINT
20,000 LB.
37,000 LB.
DEFLEC-TION IN INCHES
A 6 None None 164 108
B 6 None 6" Loose Sand & Gravel 139 110
C 6 None 6" Com-pacted Sand 88 72
D 6 None 6" Com-pacted Sand & Gravel 76 72
E 6 None 6" Crushed Stone 113 108
F 6 None None 79 79
G 8 68# Wire Mesh None 76 76
H 8 91# Wire Mesh None 69 69
J 8 159# Wire Mesh None 100 100
K 10 None None 150 150
Same as above for Section G
Offset Dowels
Same as above for Section H
0.00
0.05
0.10
DIAGRAM OF WHEEL AND DEFLECTION GAGE LOCATIONS
Free Edge
Deflection Gage
Wheel
Dummy Joints
Doweled Joint
Free Joint
40'-0"
Keyed Or Doweled Construction Joint
(See Note 3)
3"
3½"
2'-0"
2'-4"
10'-0"
20'-0"
Lane 2
20,000 Lb. Wheel Load
Lane 1
37,000 Lb. Wheel Load
NOTES
1 Shaded portion of bars show deflections 3 minutes after load had been removed.
2 A deflection on a transition slab is denoted by Tr, while a deflection on a test slab is denoted by the slab letter designation.
3 The longitudinal construction joint between lanes 1 and 2 is an undoweled keyed joint except for sections G, H and J which have a doweled butt-type construction joint.
4 All deflection measurements were taken prior to traffic.

FIGURE 4.1

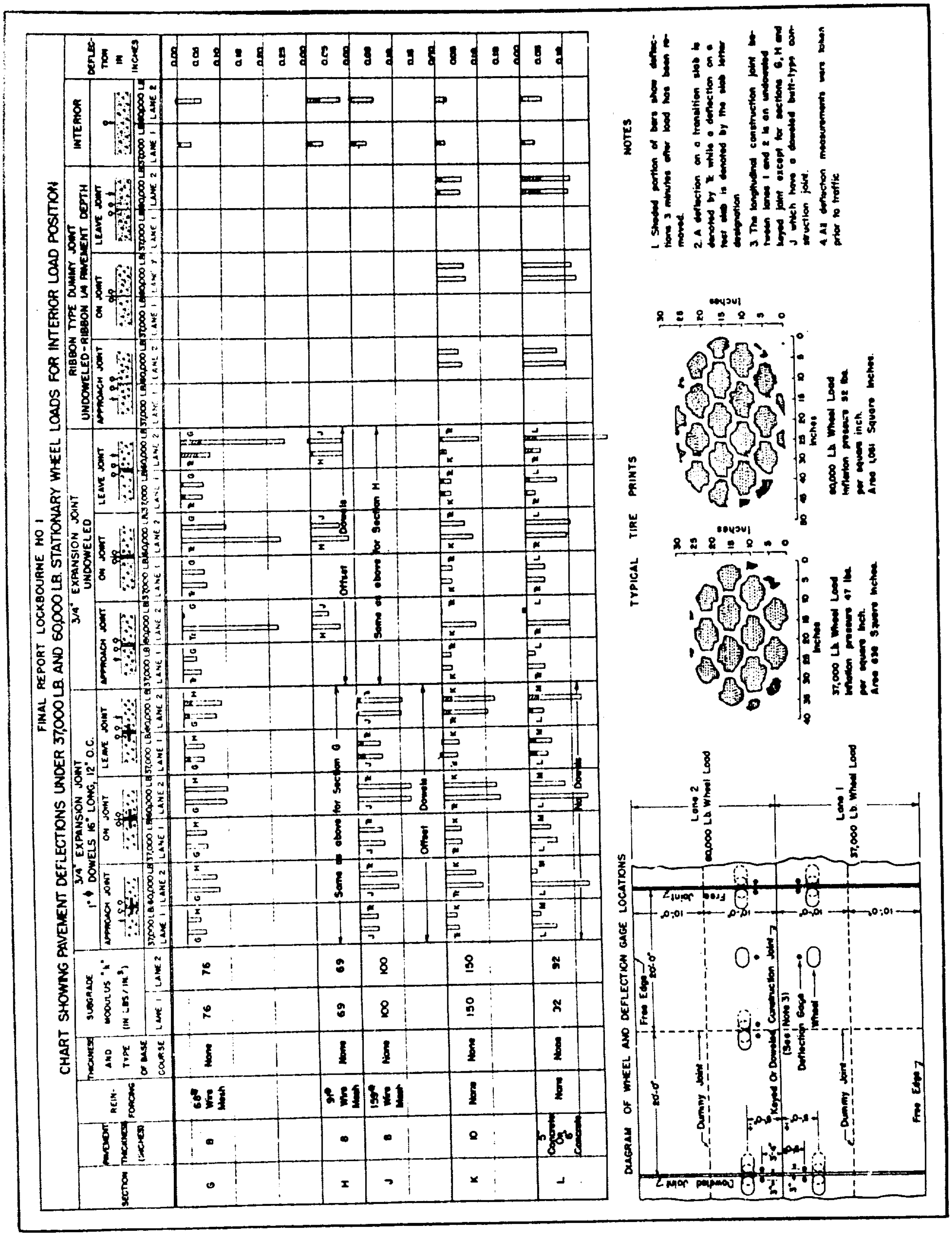

FINAL REPORT LOCKBOURNE NO I
CHART SHOWING PAVEMENT DEFLECTIONS UNDER 37,000 LB. AND 60,000 LB. STATIONARY WHEEL LOADS FOR INTERIOR LOAD POSITION
3/4" EXPANSION JOINT 1" ϕ DOWELS 16" LONG, 12" O.C.
3/4" EXPANSION JOINT UNDOWELED
RIBBON TYPE DUMMY JOINT UNDOWELED-RIBBON 1/4 PAVEMENT DEPTH
INTERIOR
APPROACH JOINT
ON JOINT
LEAVE JOINT
SECTION
PAVEMENT THICKNESS (INCHES)
REIN-FORCING
THICKNESS AND TYPE OF BASE COURSE
SUBGRADE MODULUS "k" (IN LBS/IN.³)
LANE 1
LANE 2
37,000 LB.
60,000 LB.
DEFLECTION IN INCHES
Same as above for Section G
Offset
Dowels
Same as above for Section H
No Dowels
DIAGRAM OF WHEEL AND DEFLECTION GAGE LOCATIONS
Free Edge
Dowelled Joint
Dummy Joint
Keyed Or Doweled Construction Joint
(See Note 3)
Deflection Gage
Wheel
Free Joint
Lane 2
60,000 Lb. Wheel Load
Lane 1
37,000 Lb. Wheel Load
TYPICAL TIRE PRINTS
Inches
37,000 Lb. Wheel Load
Inflation pressure 47 lbs. per square inch.
Area 638 Square inches.
60,000 Lb. Wheel Load
Inflation pressure 52 lbs. per square inch.
Area 1,061 Square inches.
NOTES
1. Shaded portion of bars show deflections 3 minutes after load has been removed.
2. A deflection on a transition slab is denoted by 'T' while a deflection on a test slab is denoted by the slab letter designation.
3. The longitudinal construction joint between lanes 1 and 2 is an undoweled keyed joint except for sections G, H and J which have a doweled butt-type construction joint.
4. All deflection measurements were taken prior to traffic.

FIGURE 4.2

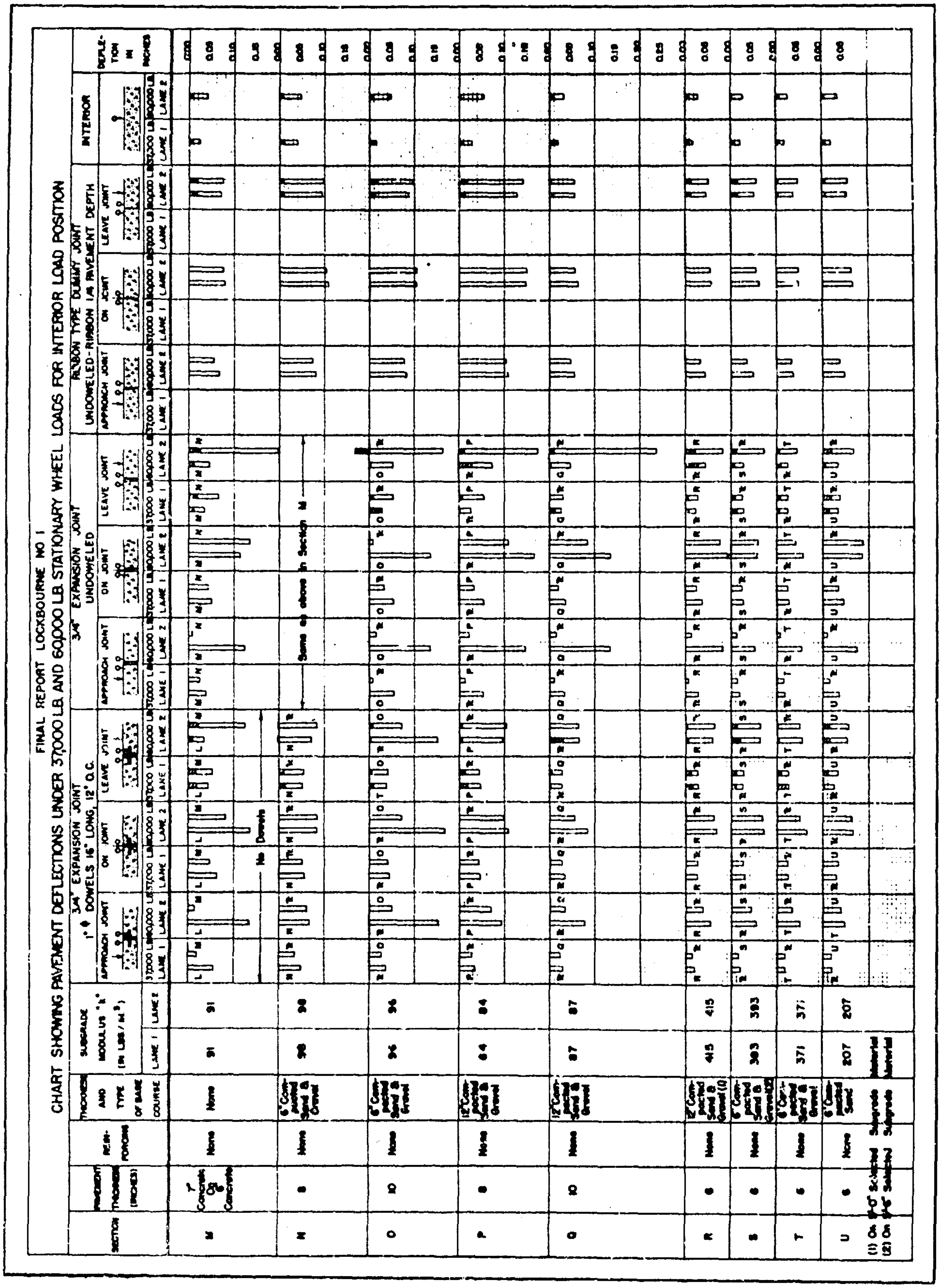

FINAL REPORT LOCKBOURNE NO 1
CHART SHOWING PAVEMENT DEFLECTIONS UNDER 37,000 LB. AND 60,000 LB. STATIONARY WHEEL LOADS FOR INTERIOR LOAD POSITION

FIGURE 4.3

FINAL REPORT LOCKBOURNE NO. 1

CHART SHOWING PAVEMENT DEFLECTIONS UNDER 37,000 LB. AND 60,000 LB. STATIONARY WHEEL LOADS FOR EDGE LOAD POSITION

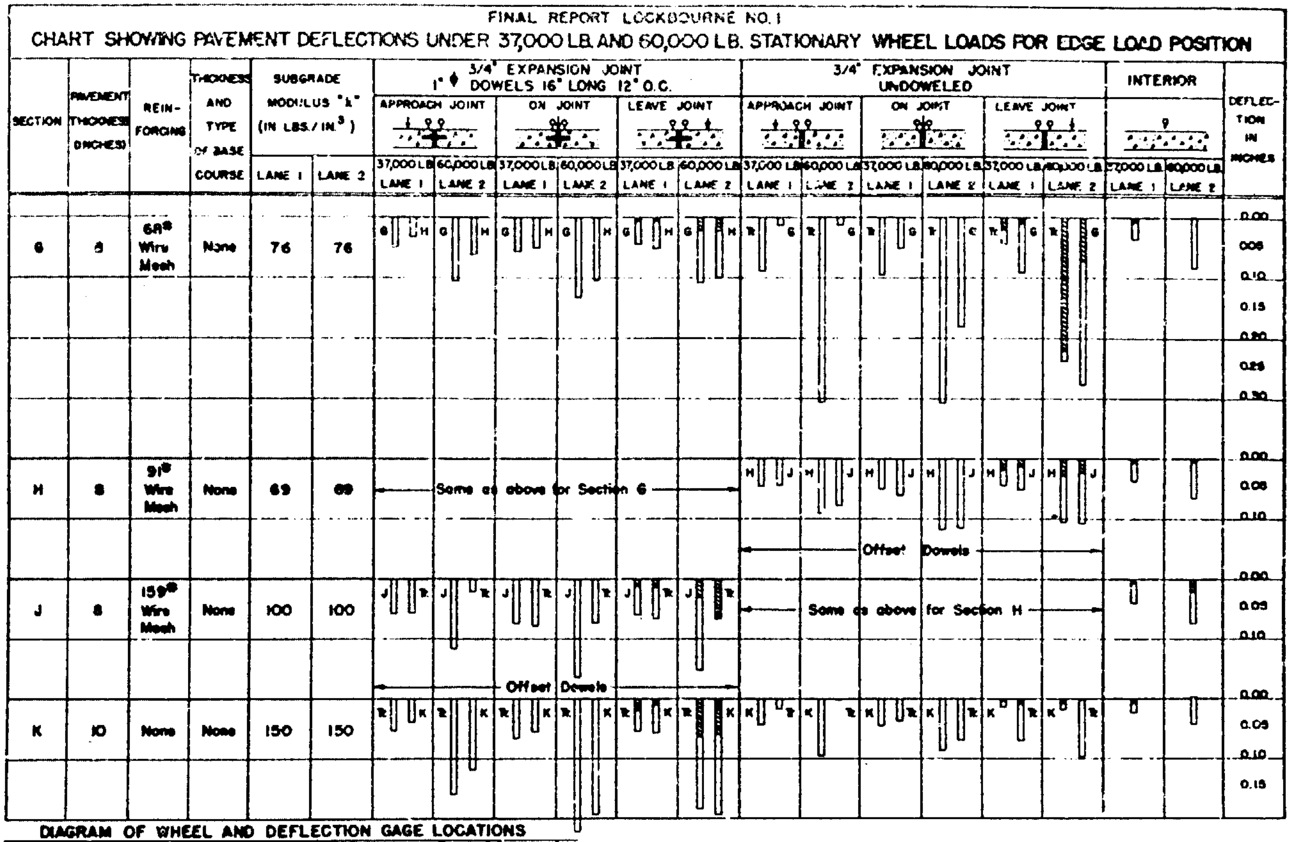
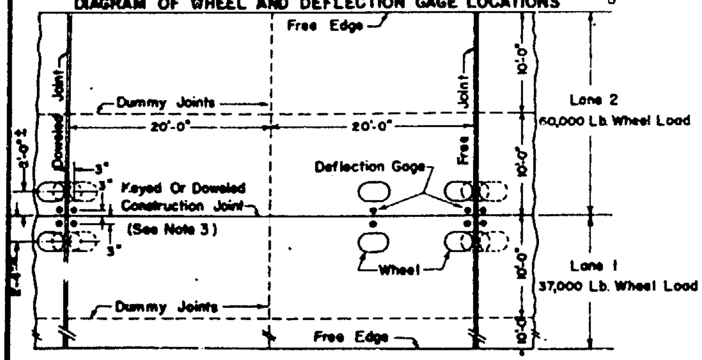


DIAGRAM OF WHEEL AND DEFLECTION GAGE LOCATIONS



NOTES

1 Shaded portion of bars show deflections 3 minutes after load had been removed.
2 A deflection on a transition slab is denoted by Tr, while a deflection on a test slab is denoted by the slab letter designation.
3 The longitudinal construction joint between lanes 1 and 2 is an undoweled keyed joint except for sections G, H and J which have a doweled butt-type construction joint.
4 All deflection measurements were taken prior to traffic.

FIGURE 4.4

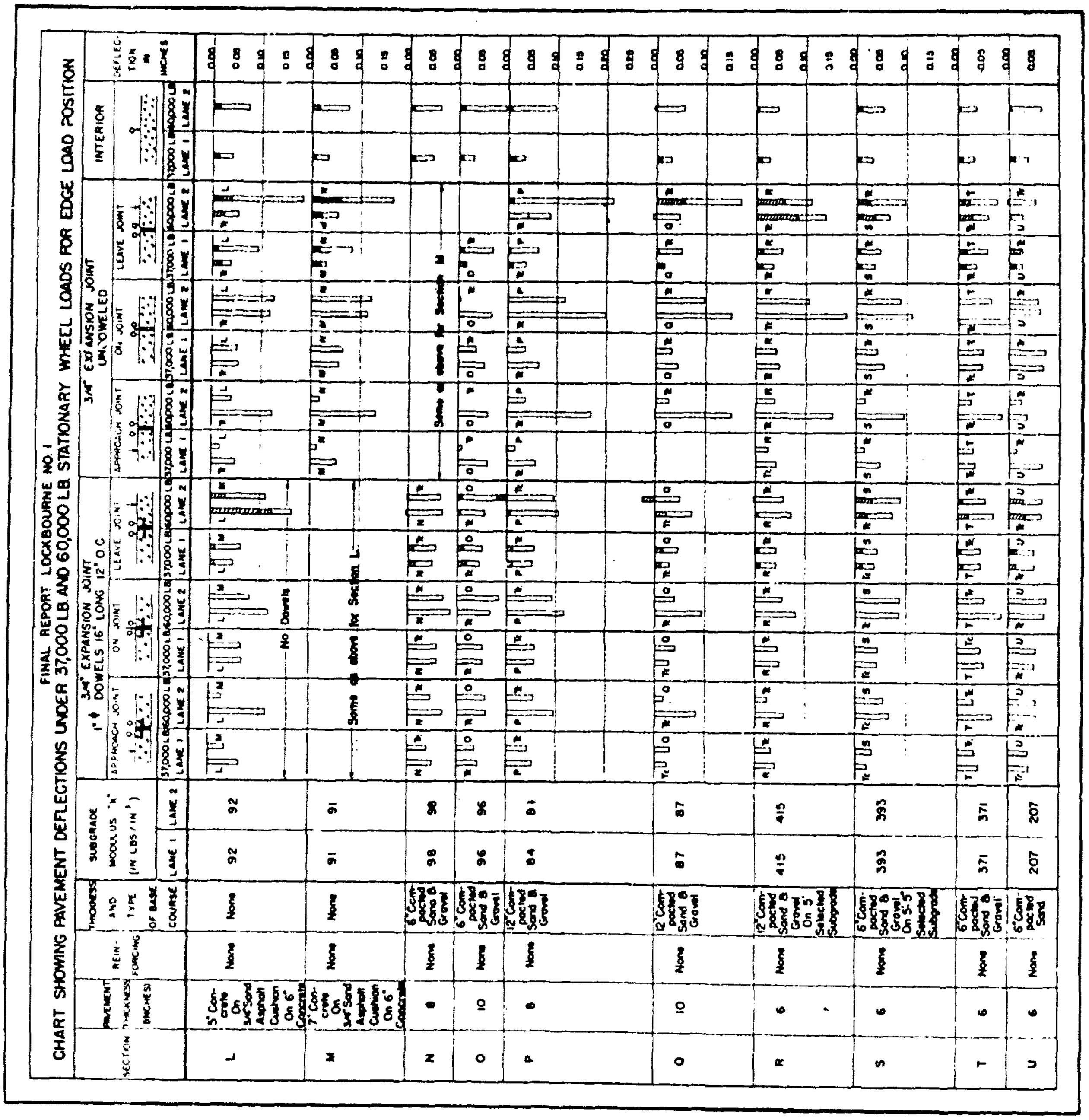
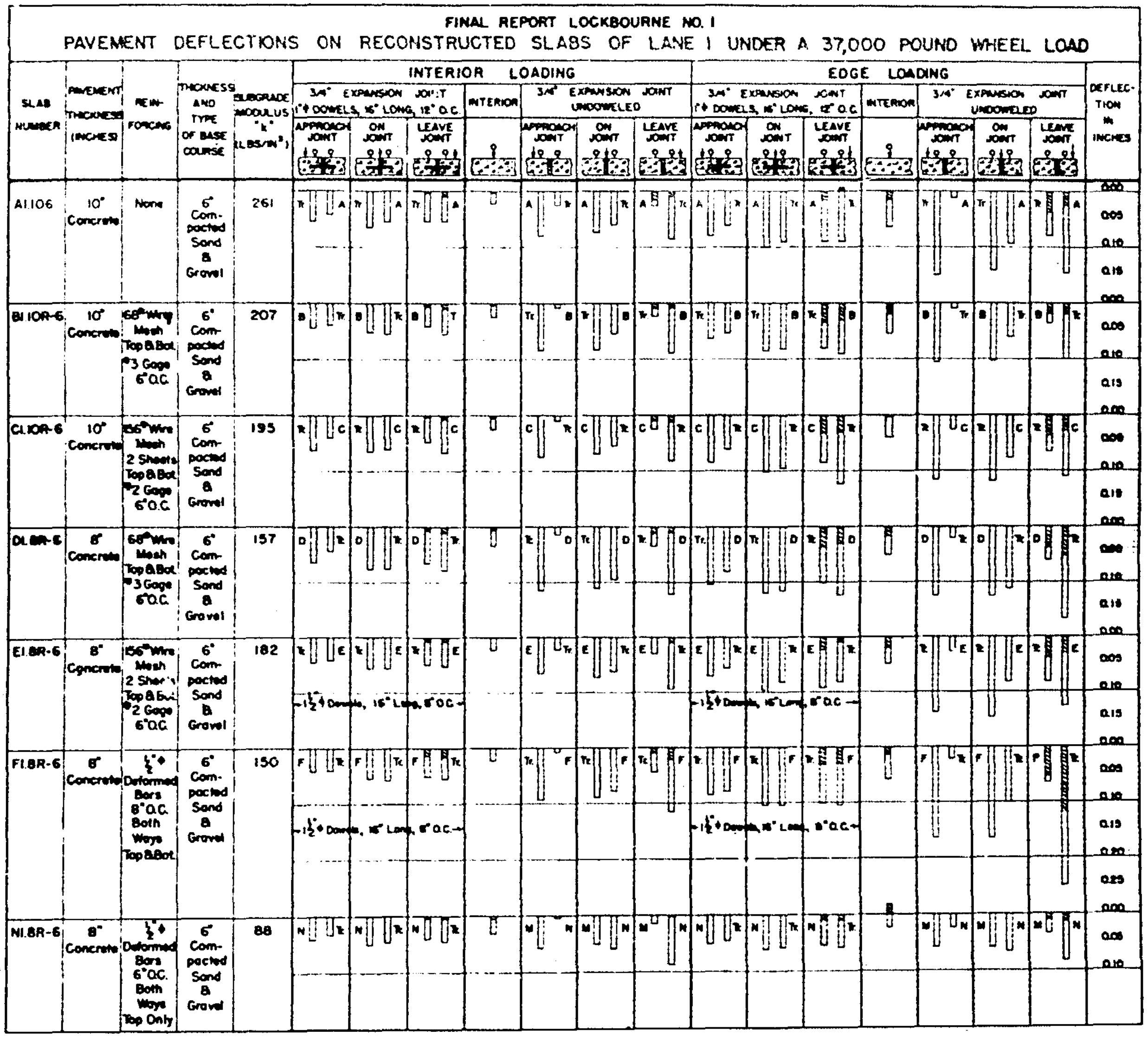

FINAL REPORT LOCKBOURNE NO. I
CHART SHOWING PAVEMENT DEFLECTIONS UNDER 37,000 LB. AND 60,000 LB. STATIONARY WHEEL LOADS FOR EDGE LOAD POSITION

FIGURE 4.5



NOTES:
1. See Figure 4.4 for diagram of wheel and deflection gage location for the 37,000 pound wheel load in lane 1.
2. The longitudinal construction joint between lanes 1 and 2 is a free joint
3. Notes 1, 2, and 4 of Figure 4.4 apply to this sheet.

FIGURE 4.6

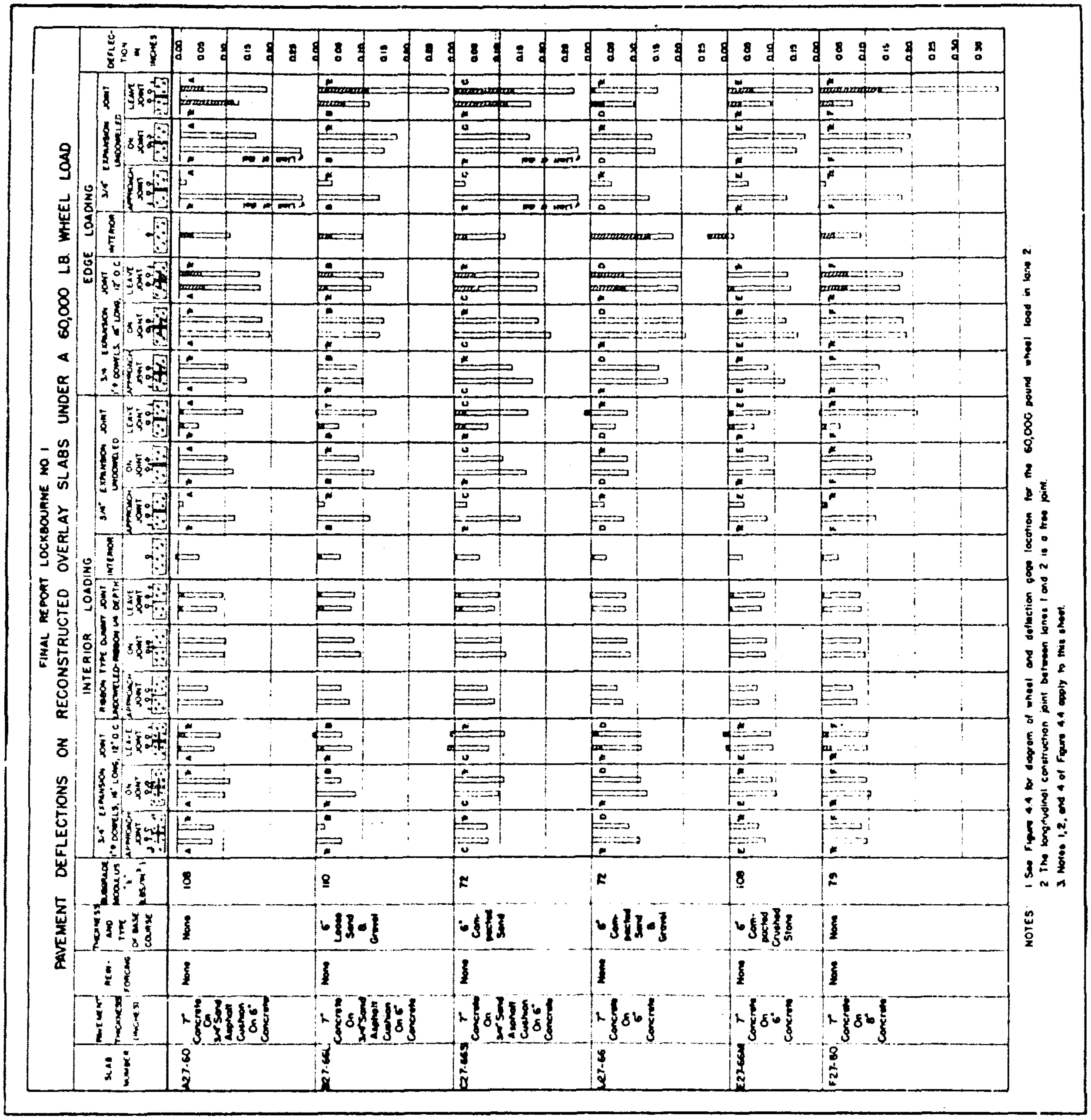

FINAL REPORT LOCKBOURNE NO 1
PAVEMENT DEFLECTIONS ON RECONSTRUCTED OVERLAY SLABS UNDER A 60,000 LB. WHEEL LOAD
INTERIOR LOADING
EDGE LOADING
NOTES
1 See Figure 4.4 for diagram of wheel and deflection gage location for the 60,000 pound wheel load in lane 2.
2 The longitudinal construction joint between lanes 1 and 2 is a free joint.
3. Notes 1,2, and 4 of Figure 4.4 apply to this sheet.

FIGURE 4.7

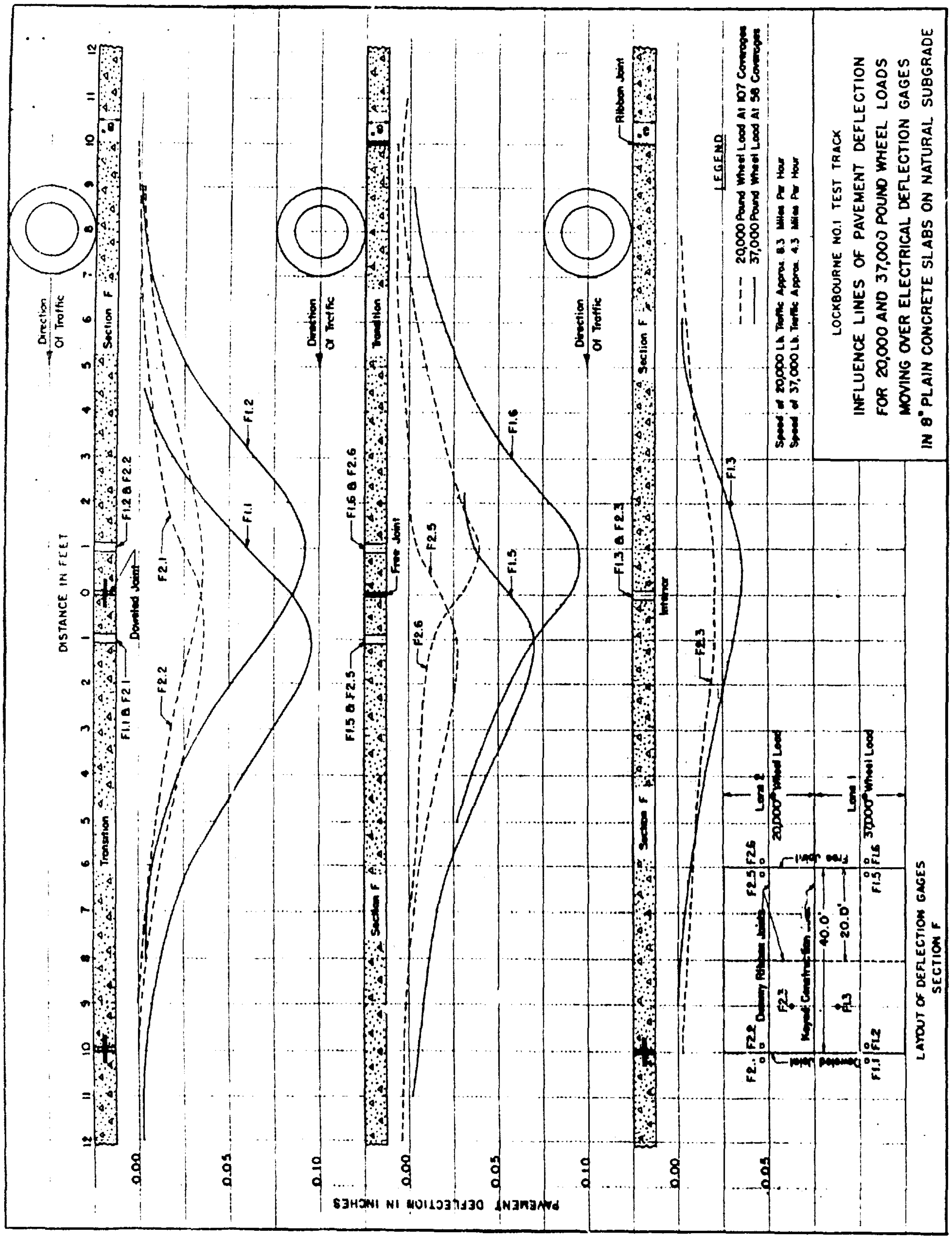

LOCKBOURNE NO.1 TEST TRACK
INFLUENCE LINES OF PAVEMENT DEFLECTION
FOR 20,000 AND 37,000 POUND WHEEL LOADS
MOVING OVER ELECTRICAL DEFLECTION GAGES
IN 8" PLAIN CONCRETE SLABS ON NATURAL SUBGRADE
LEGEND
20,000 Pound Wheel Load At 107 Coverages
37,000 Pound Wheel Load At 58 Coverages
Speed of 20,000 Lb. Traffic Approx. 8.3 Miles Per Hour
Speed of 37,000 Lb. Traffic Approx. 4.3 Miles Per Hour
DISTANCE IN FEET
PAVEMENT DEFLECTION IN INCHES
Direction Of Traffic
Section F
Transition
Doweled Joint
Free Joint
Ribbon Joint
Interior
LAYOUT OF DEFLECTION GAGES
SECTION F
Lane 1
Lane 2
20,000# Wheel Load
37,000# Wheel Load
40.0'
20.0'

FIGURE 4.8

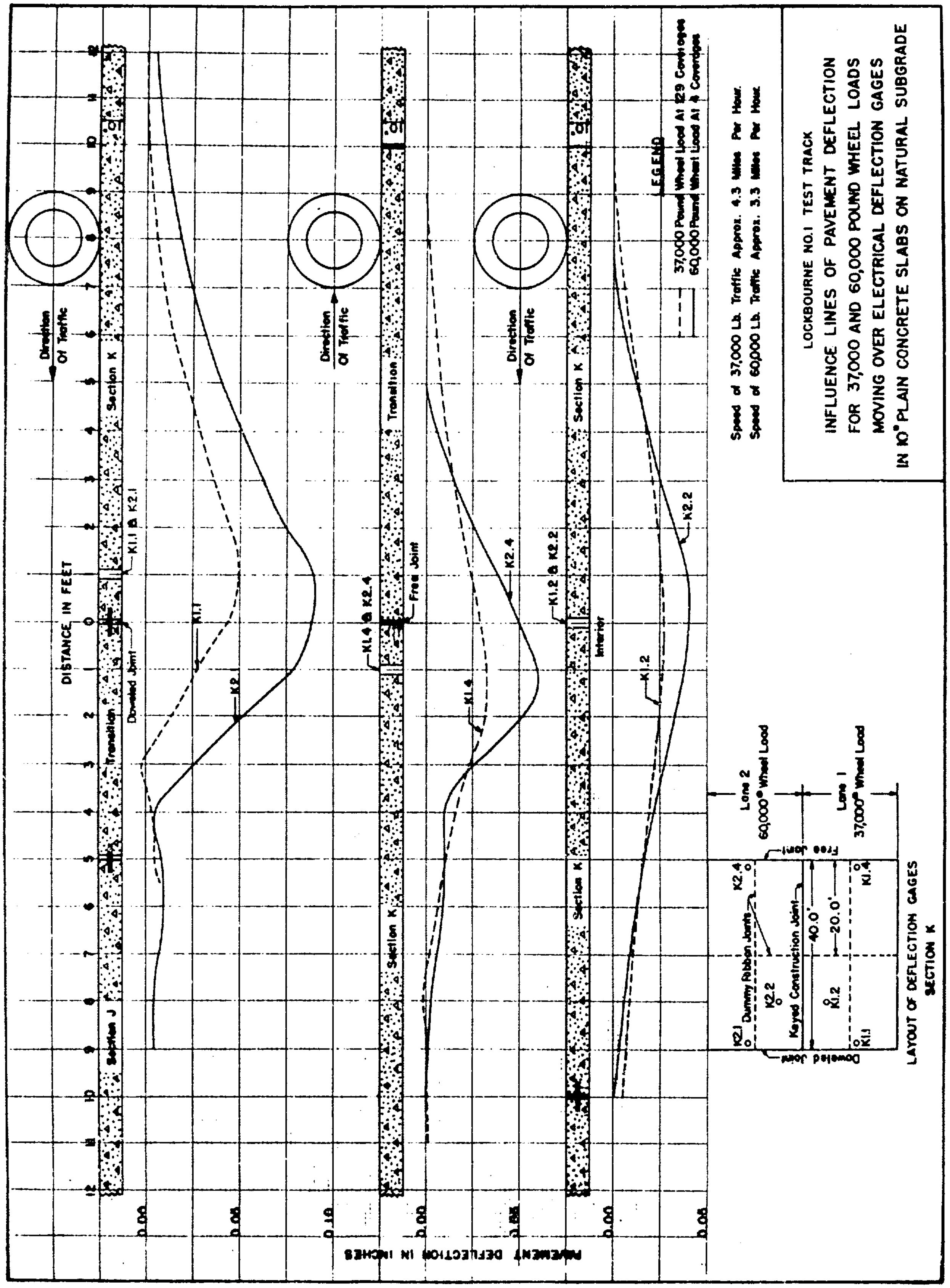

LOCKBOURNE NO. I TEST TRACK
INFLUENCE LINES OF PAVEMENT DEFLECTION
FOR 37,000 AND 60,000 POUND WHEEL LOADS
MOVING OVER ELECTRICAL DEFLECTION GAGES
IN 10" PLAIN CONCRETE SLABS ON NATURAL SUBGRADE
LEGEND
37,000 Pound Wheel Load At 129 Coverages
60,000 Pound Wheel Load At 4 Coverages
Speed of 37,000 Lb. Traffic Approx. 4.3 Miles Per Hour.
Speed of 60,000 Lb. Traffic Approx. 3.3 Miles Per Hour.
DISTANCE IN FEET
PAVEMENT DEFLECTION IN INCHES
Direction Of Traffic
Section J
Section K
Transition
Doweled Joint
Free Joint
Interior
K1.1 & K2.1
K1.4 & K2.4
K1.2 & K2.2
LAYOUT OF DEFLECTION GAGES
SECTION K
Lane 2
60,000# Wheel Load
Lane 1
37,000# Wheel Load
Dummy Ribbon Joints
Keyed Construction Joint
40.0'
20.0'

FIGURE 4.9

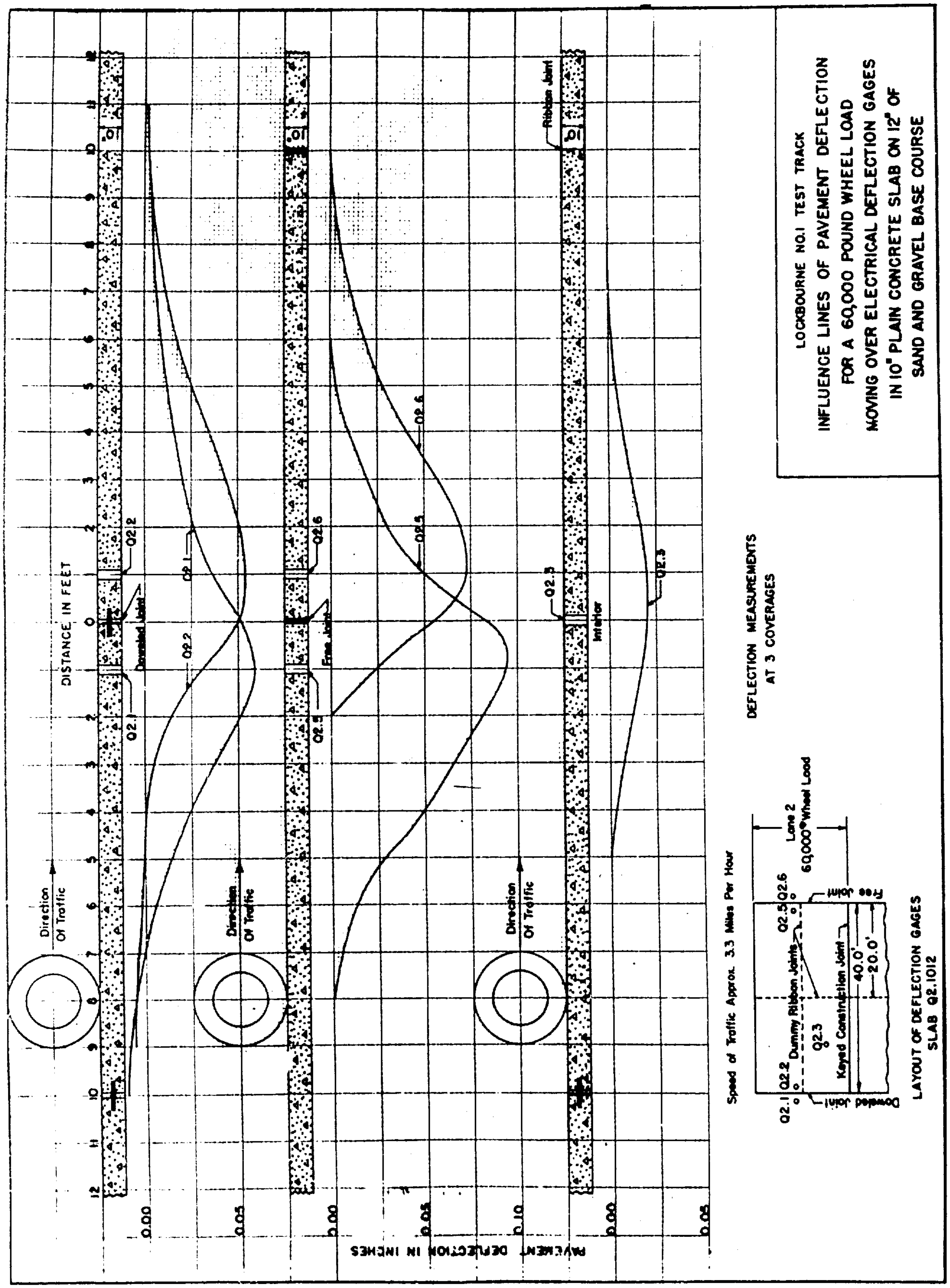

LOCKBOURNE NO.1 TEST TRACK
INFLUENCE LINES OF PAVEMENT DEFLECTION
FOR A 60,000 POUND WHEEL LOAD
MOVING OVER ELECTRICAL DEFLECTION GAGES
IN 10" PLAIN CONCRETE SLAB ON 12" OF
SAND AND GRAVEL BASE COURSE
DISTANCE IN FEET
PAVEMENT DEFLECTION IN INCHES
Direction Of Traffic
Doweled Joint
Free Joint
Ribbon Joint
Interior
Q2.1
Q2.2
Q2.3
Q2.5
Q2.6
DEFLECTION MEASUREMENTS
AT 3 COVERAGES
Speed of Traffic Approx. 3.3 Miles Per Hour
Lane 2
60,000# Wheel Load
Dummy Ribbon Joints
Keyed Construction Joint
40.0'
20.0'
LAYOUT OF DEFLECTION GAGES
SLAB Q2.1012

FIGURE 4.10

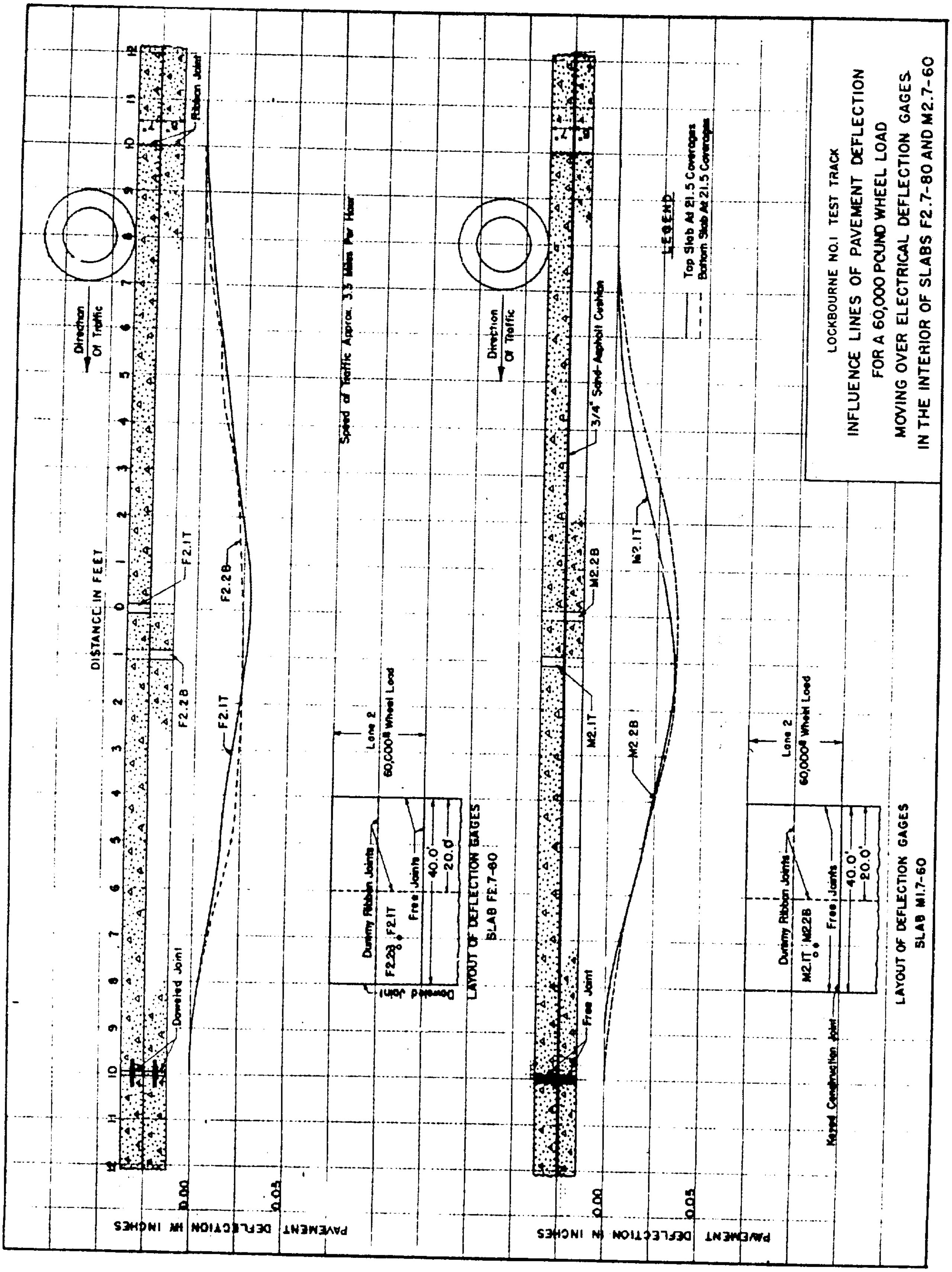

LOCKBOURNE NO. 1 TEST TRACK
INFLUENCE LINES OF PAVEMENT DEFLECTION
FOR A 60,000 POUND WHEEL LOAD
MOVING OVER ELECTRICAL DEFLECTION GAGES
IN THE INTERIOR OF SLABS F2.7-80 AND M2.7-60
LEGEND
Top Slab At 21.5 Coverages
Bottom Slab At 21.5 Coverages
DISTANCE IN FEET
PAVEMENT DEFLECTION IN INCHES
Direction Of Traffic
Speed of Traffic Approx. 3.5 Miles Per Hour
Ribbon Joint
Doweled Joint
Free Joint
3/4" Sand-Asphalt Cushion
Keyed Construction Joint
Dummy Ribbon Joints
Free Joints
Lane 2
60,000# Wheel Load
LAYOUT OF DEFLECTION GAGES
SLAB FE.7-60
LAYOUT OF DEFLECTION GAGES
SLAB M1.7-60

FIGURE 4.11

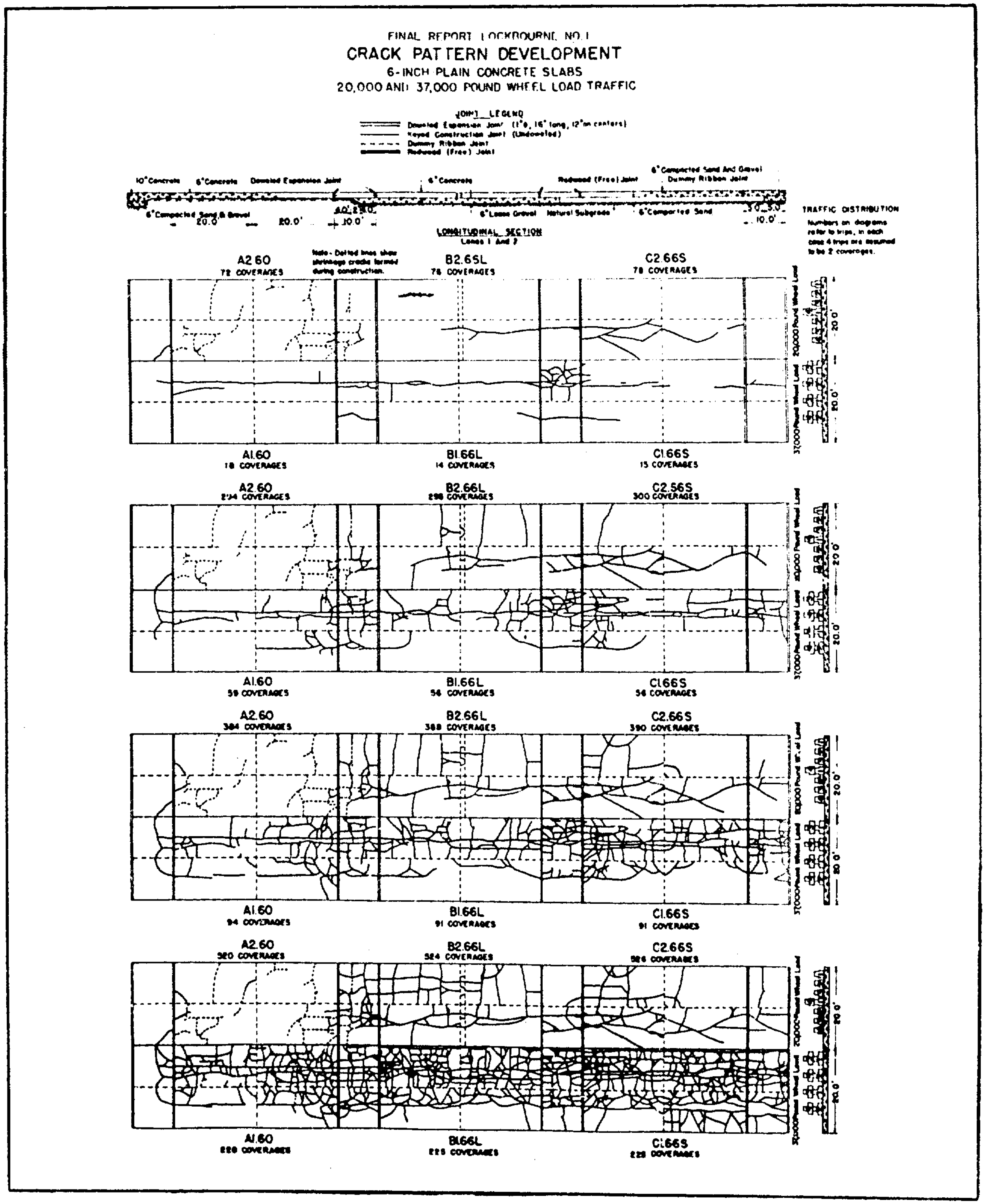

FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
6-INCH PLAIN CONCRETE SLABS
20,000 AND 37,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
A2.60
B2.66L
C2.66S
A1.60
B1.66L
C1.66S

FIGURE 5.0

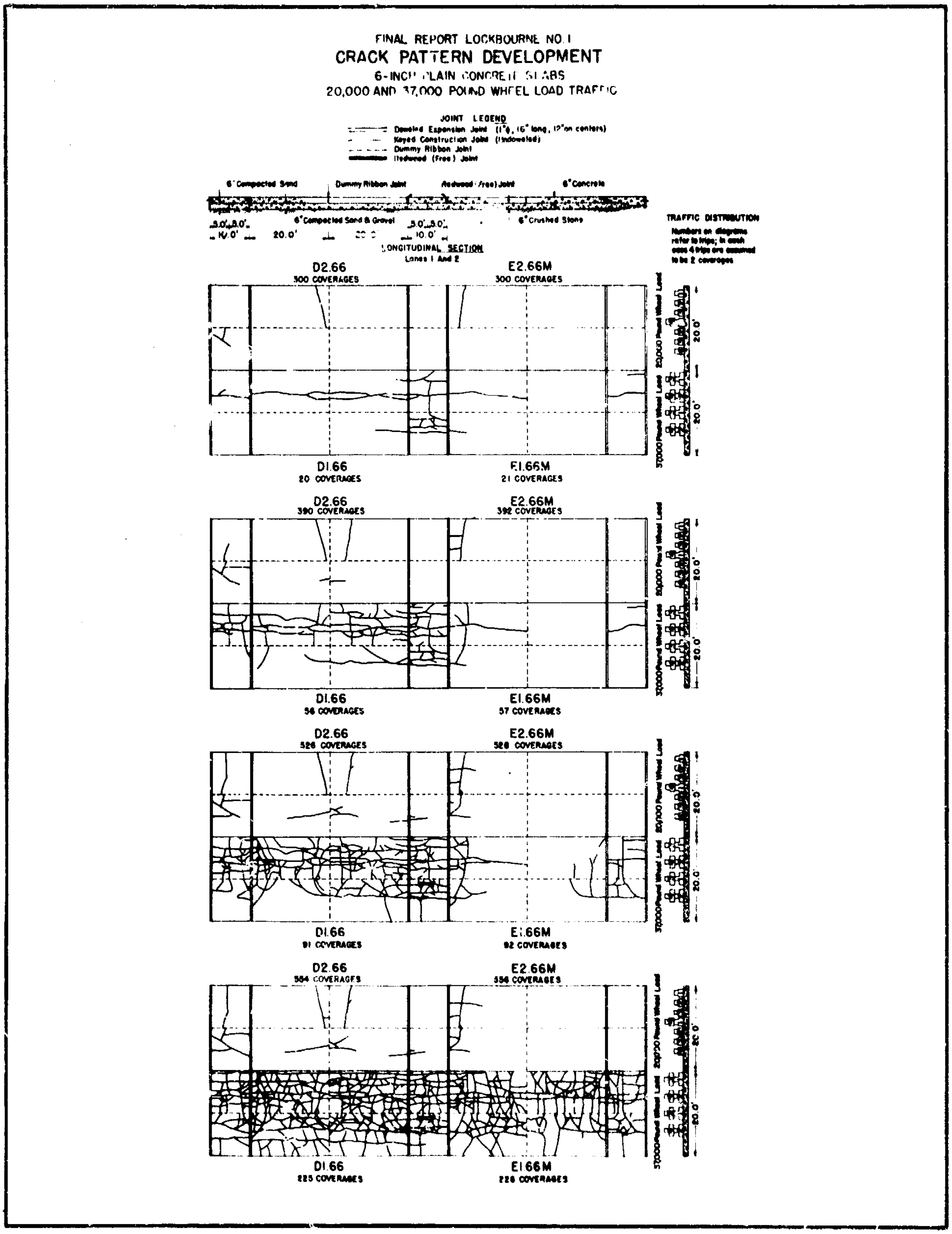

FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
6-INCH PLAIN CONCRETE SLABS
20,000 AND 37,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
Doweled Expansion Joint (1"ϕ, 16" long, 12" on centers)
Keyed Construction Joint (Undoweled)
Dummy Ribbon Joint
Redwood (Free) Joint
6" Compacted Sand
Dummy Ribbon Joint
Redwood (Free) Joint
6" Concrete
6" Compacted Sand & Gravel
6" Crushed Stone
LONGITUDINAL SECTION
Lanes 1 And 2
TRAFFIC DISTRIBUTION
Numbers on diagrams refer to trips; in each case 4 trips are assumed to be 2 coverages
D2.66
300 COVERAGES
E2.66M
300 COVERAGES
D1.66
20 COVERAGES
E1.66M
21 COVERAGES
D2.66
390 COVERAGES
E2.66M
392 COVERAGES
D1.66
56 COVERAGES
E1.66M
57 COVERAGES
D2.66
526 COVERAGES
E2.66M
528 COVERAGES
D1.66
91 COVERAGES
E1.66M
92 COVERAGES
D2.66
554 COVERAGES
E2.66M
556 COVERAGES
D1.66
225 COVERAGES
E1.66M
226 COVERAGES
20,000 Pound Wheel Load
37,000 Pound Wheel Load
20.0'

FIGURE 5.1

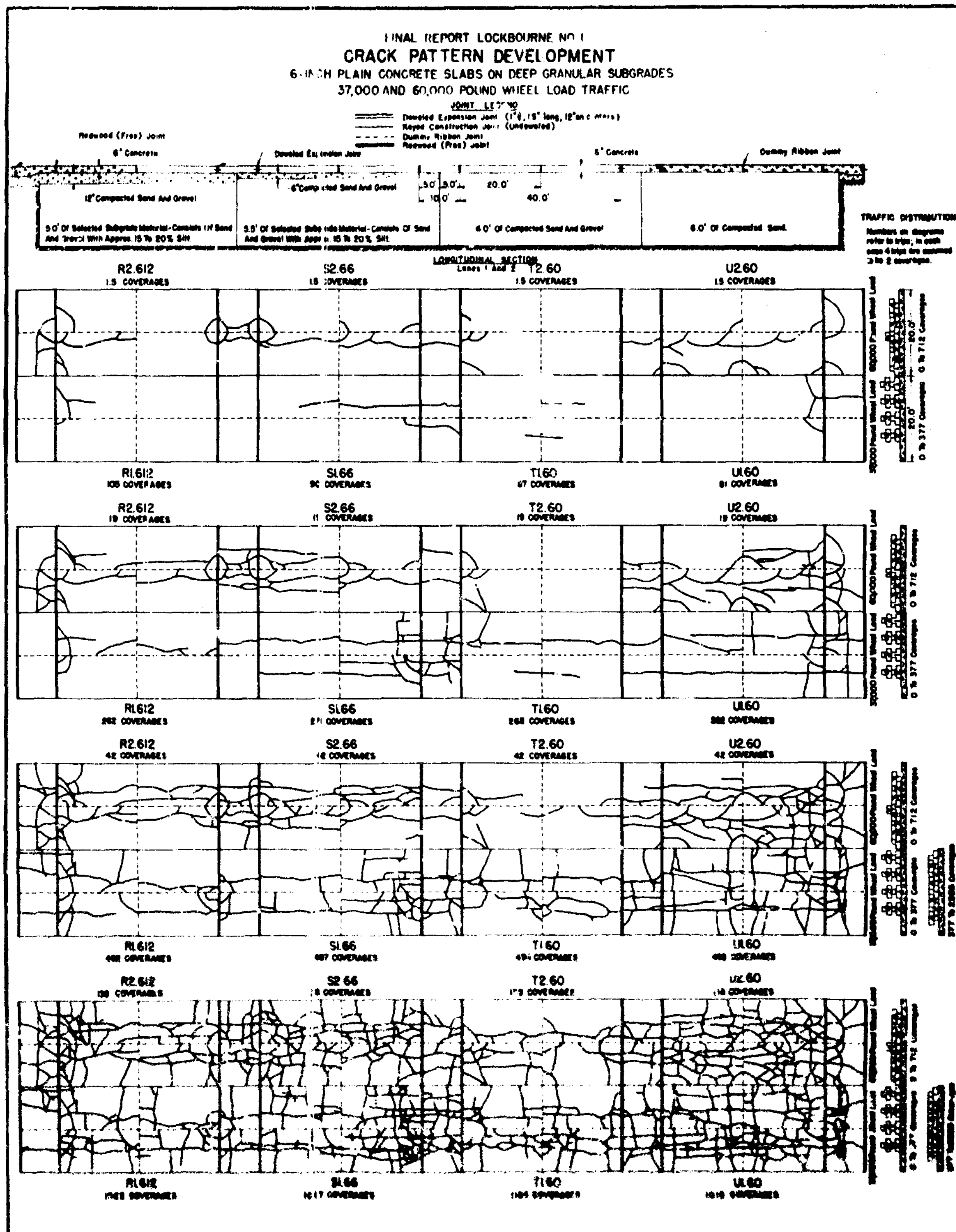
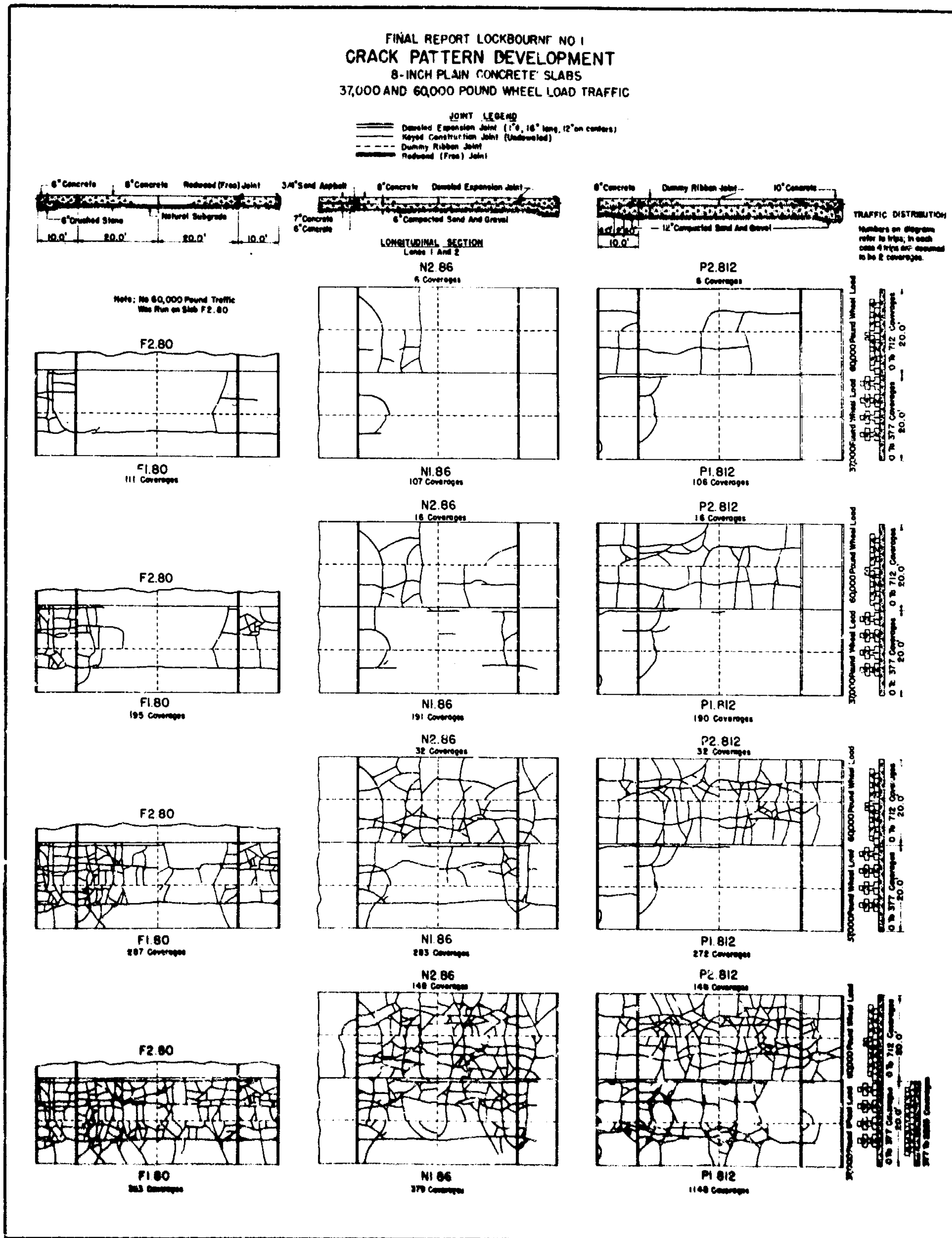

FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
6-INCH PLAIN CONCRETE SLABS ON DEEP GRANULAR SUBGRADES
37,000 AND 60,000 POUND WHEEL LOAD TRAFFIC
LONGITUDINAL SECTION
R2.612
S2.66
T2.60
U2.60
RL612
SL66
TL60
UL60
TRAFFIC DISTRIBUTION

FIGURE 5.2



FINAL REPORT LOCKBOURNE NO 1
CRACK PATTERN DEVELOPMENT
8-INCH PLAIN CONCRETE SLABS
37,000 AND 60,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
Note: No 60,000 Pound Traffic Was Run on Slab F2.80
F2.80
F1.80
111 Coverages
N2.86
6 Coverages
N1.86
107 Coverages
P2.812
6 Coverages
P1.812
106 Coverages
F1.80
195 Coverages
N2.86
16 Coverages
N1.86
191 Coverages
P2.812
16 Coverages
P1.812
190 Coverages
F1.80
287 Coverages
N2.86
32 Coverages
N1.86
283 Coverages
P2.812
32 Coverages
P1.812
272 Coverages
F1.80
N2.86
148 Coverages
N1.86
379 Coverages
P2.812
148 Coverages
P1.812
1148 Coverages

FIGURE 5.3

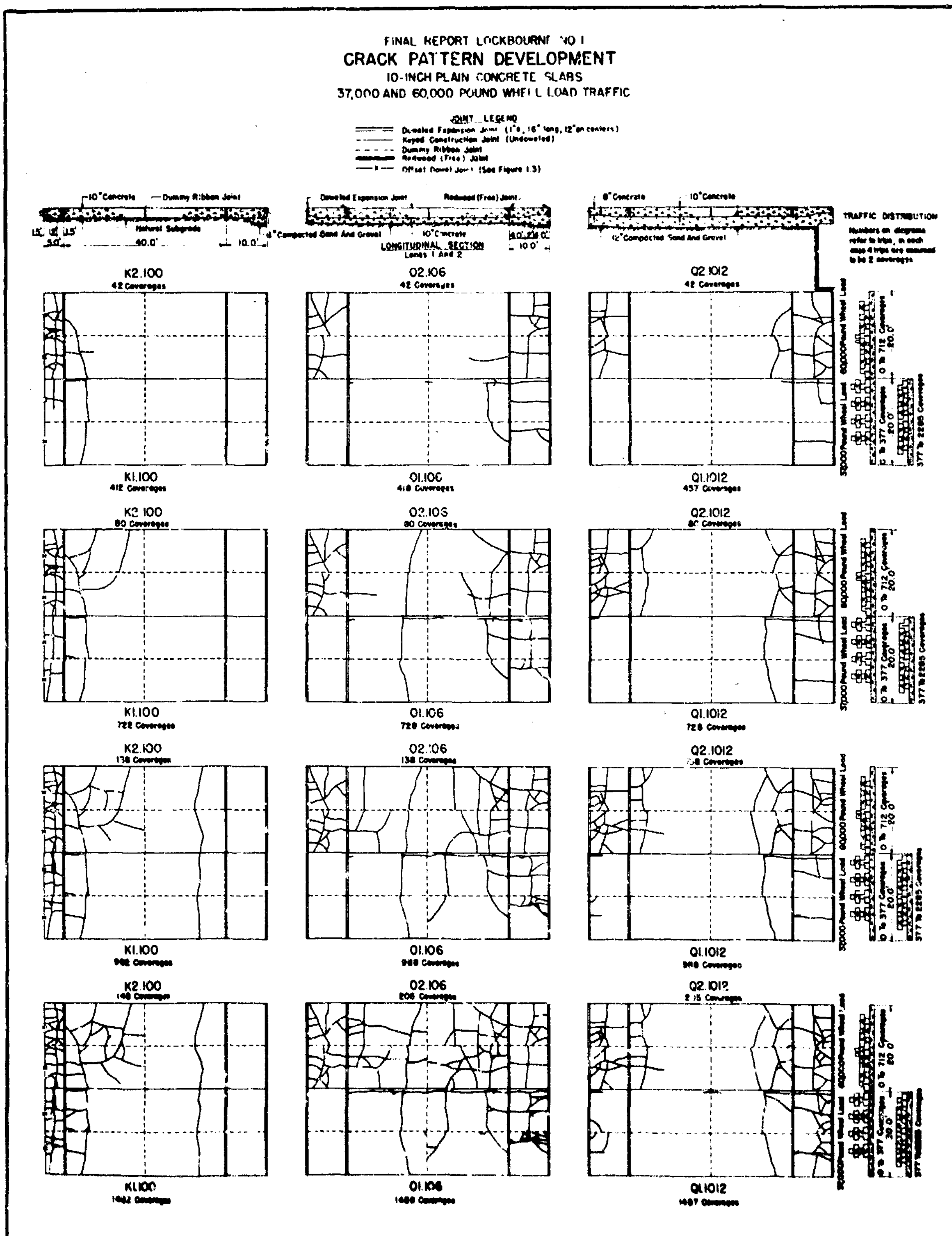

FINAL REPORT LOCKBOURNE NO 1
CRACK PATTERN DEVELOPMENT
10-INCH PLAIN CONCRETE SLABS
37,000 AND 60,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
K2.100
02.106
Q2.1012
K1.100
01.106
Q1.1012

FIGURE 5.4

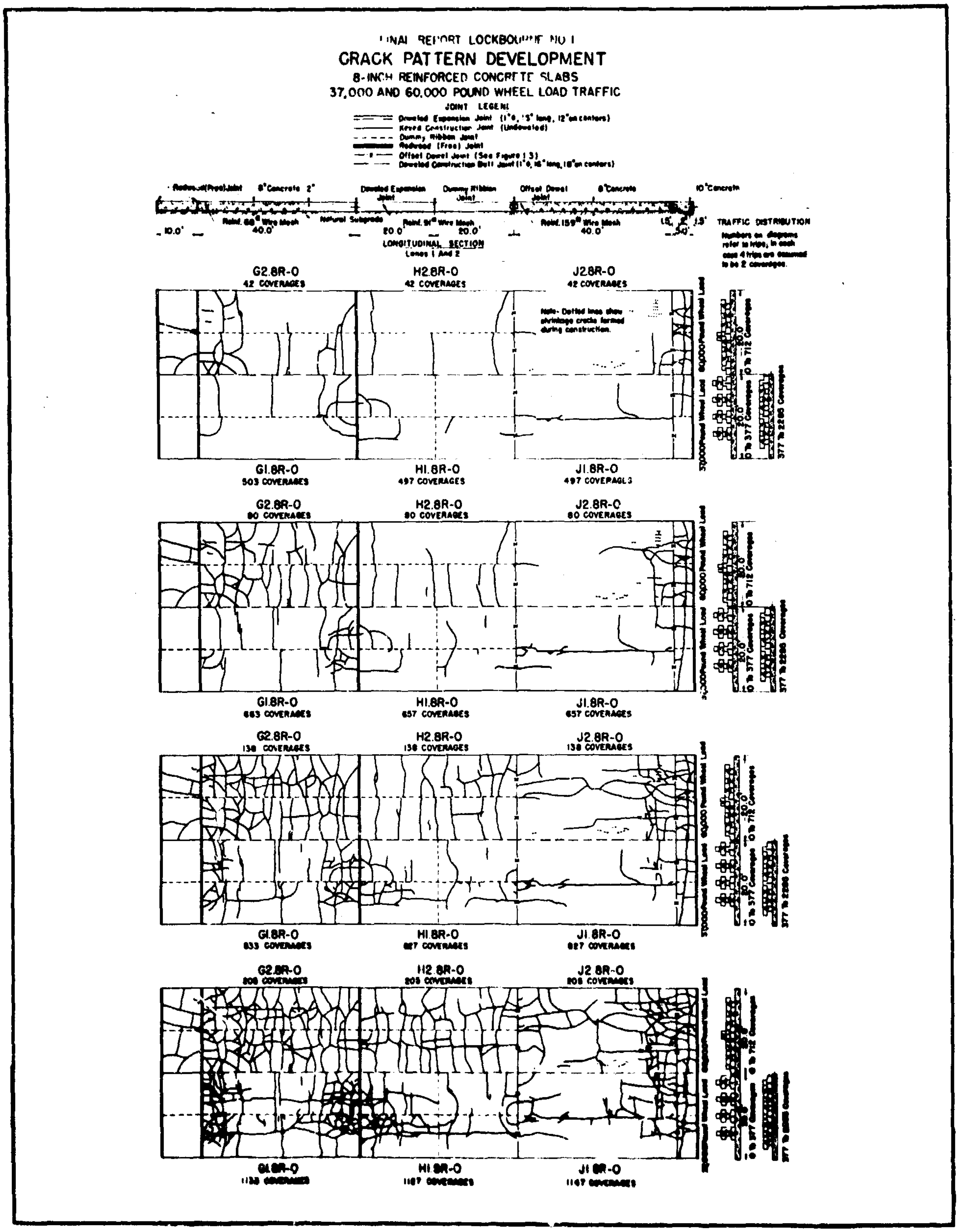
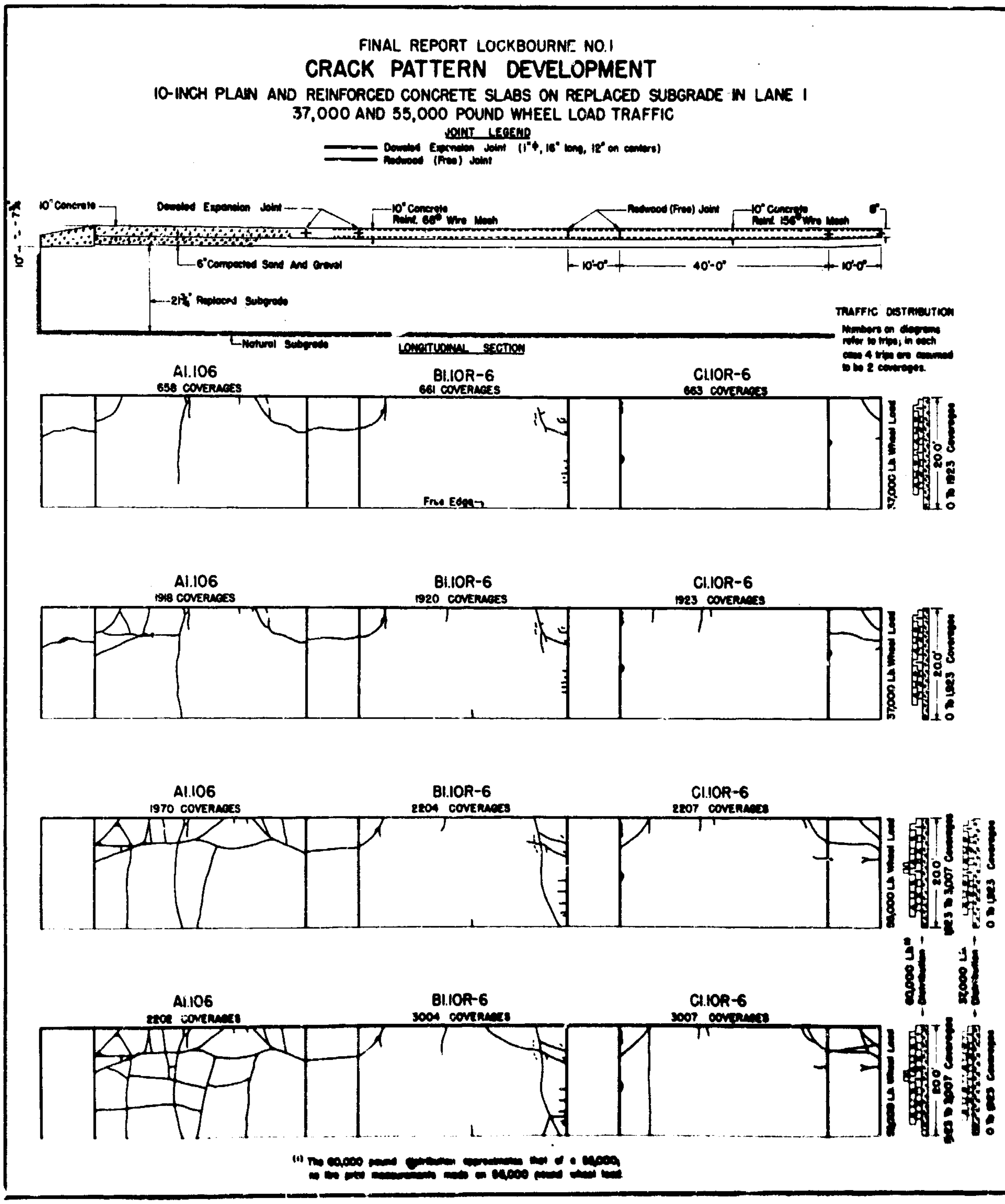

FINAL REPORT LOCKBOURNE NO 1
CRACK PATTERN DEVELOPMENT
8-INCH REINFORCED CONCRETE SLABS
37,000 AND 60,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
LONGITUDINAL SECTION
Lanes 1 And 2
TRAFFIC DISTRIBUTION
G2.8R-0
H2.8R-0
J2.8R-0
G1.8R-0
H1.8R-0
J1.8R-0

FIGURE 5.8



FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
10-INCH PLAIN AND REINFORCED CONCRETE SLABS ON REPLACED SUBGRADE IN LANE I
37,000 AND 55,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
Redwood (Free) Joint
10" Concrete
Doweled Expansion Joint
6" Compacted Sand And Gravel
Replaced Subgrade
Natural Subgrade
LONGITUDINAL SECTION
10'-0"
40'-0"
TRAFFIC DISTRIBUTION
A1.106
658 COVERAGES
B1.10R-6
661 COVERAGES
C1.10R-6
663 COVERAGES
Free Edge
A1.106
1918 COVERAGES
B1.10R-6
1920 COVERAGES
C1.10R-6
1923 COVERAGES
A1.106
1970 COVERAGES
B1.10R-6
2204 COVERAGES
C1.10R-6
2207 COVERAGES
A1.106
2202 COVERAGES
B1.10R-6
3004 COVERAGES
C1.10R-6
3007 COVERAGES

FIGURE 5.6

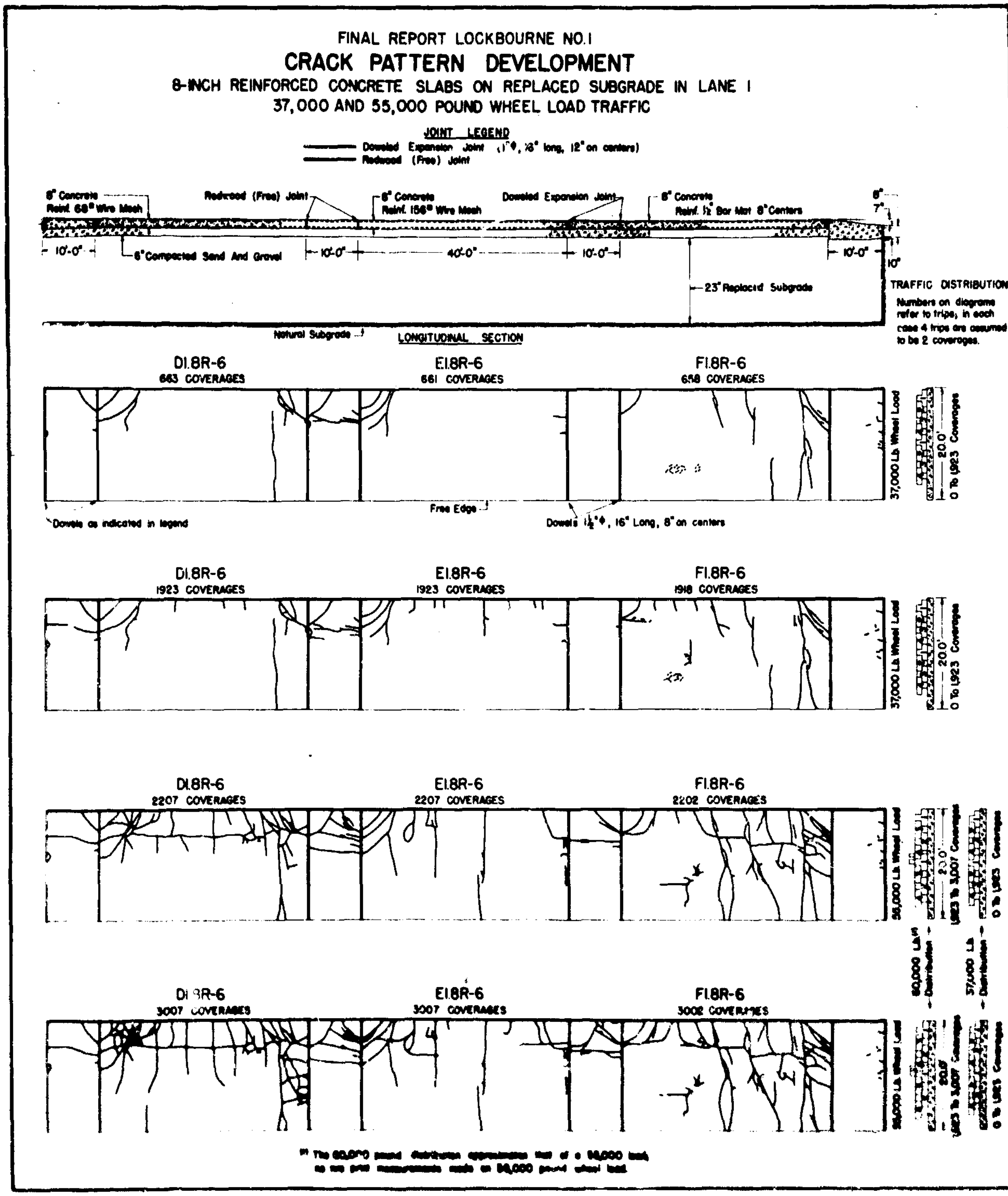
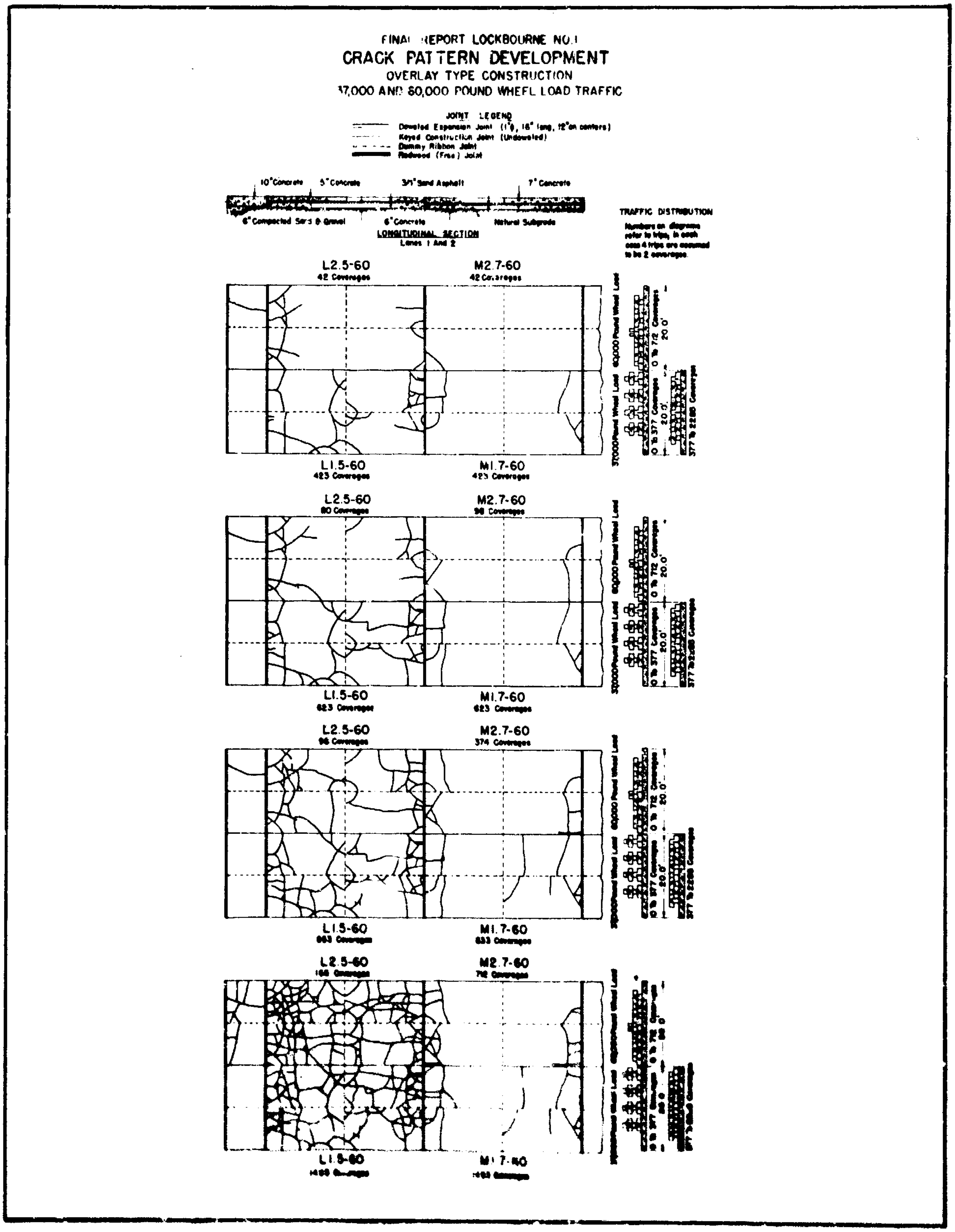
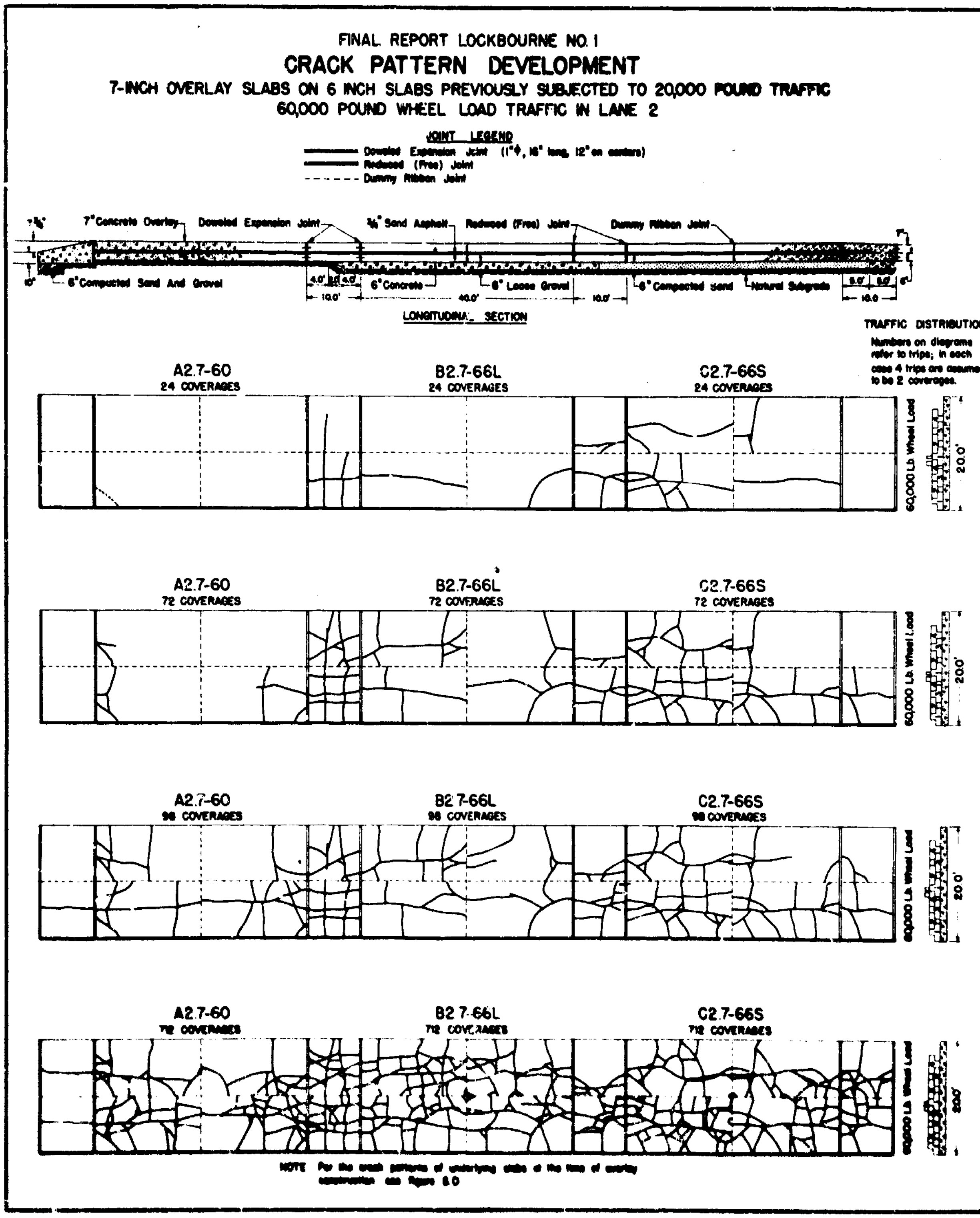

FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
8-INCH REINFORCED CONCRETE SLABS ON REPLACED SUBGRADE IN LANE I
37,000 AND 55,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
Redwood (Free) Joint
Doweled Expansion Joint
6" Compacted Sand And Gravel
23" Replaced Subgrade
Natural Subgrade
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
D1.8R-6
E1.8R-6
F1.8R-6
1923 COVERAGES
2207 COVERAGES
3007 COVERAGES
Free Edge
Dowels as indicated in legend

FIGURE 5.7



FINAL REPORT LOCKBOURNE NO.1
CRACK PATTERN DEVELOPMENT
OVERLAY TYPE CONSTRUCTION
37,000 AND 60,000 POUND WHEEL LOAD TRAFFIC
JOINT LEGEND
Doweled Expansion Joint (1"φ, 16" long, 12" on centers)
Keyed Construction Joint (Undoweled)
Dummy Ribbon Joint
Redwood (Free) Joint
10" Concrete
5" Concrete
3/4" Sand Asphalt
7" Concrete
6" Compacted Sand & Gravel
6" Concrete
Natural Subgrade
LONGITUDINAL SECTION
Lanes 1 And 2
TRAFFIC DISTRIBUTION
L2.5-60
42 Coverages
M2.7-60
42 Coverages
L1.5-60
423 Coverages
M1.7-60
423 Coverages
L2.5-60
80 Coverages
M2.7-60
98 Coverages
L1.5-60
623 Coverages
M1.7-60
623 Coverages
L2.5-60
96 Coverages
M2.7-60
374 Coverages
L1.5-60
M1.7-60
L2.5-60
M2.7-60
L1.5-60

FIGURE 5.8



FINAL REPORT LOCKBOURNE NO. 1
CRACK PATTERN DEVELOPMENT
7-INCH OVERLAY SLABS ON 6 INCH SLABS PREVIOUSLY SUBJECTED TO 20,000 POUND TRAFFIC
60,000 POUND WHEEL LOAD TRAFFIC IN LANE 2
JOINT LEGEND
Redwood (Free) Joint
Dummy Ribbon Joint
7" Concrete Overlay
Doweled Expansion Joint
Sand Asphalt
Redwood (Free) Joint
Dummy Ribbon Joint
6" Compacted Sand And Gravel
6" Concrete
6" Loose Gravel
6" Compacted Sand
Natural Subgrade
10.0'
40.0'
10.0'
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
A2.7-60
24 COVERAGES
B2.7-66L
24 COVERAGES
C2.7-66S
24 COVERAGES
60,000 Lb. Wheel Load
20.0'
A2.7-60
72 COVERAGES
B2.7-66L
72 COVERAGES
C2.7-66S
72 COVERAGES
60,000 Lb. Wheel Load
20.0'
A2.7-60
98 COVERAGES
B2.7-66L
98 COVERAGES
C2.7-66S
98 COVERAGES
60,000 Lb. Wheel Load
20.0'
A2.7-60
712 COVERAGES
B2.7-66L
712 COVERAGES
C2.7-66S
712 COVERAGES
60,000 Lb. Wheel Load
20.0'

FIGURE 5.9

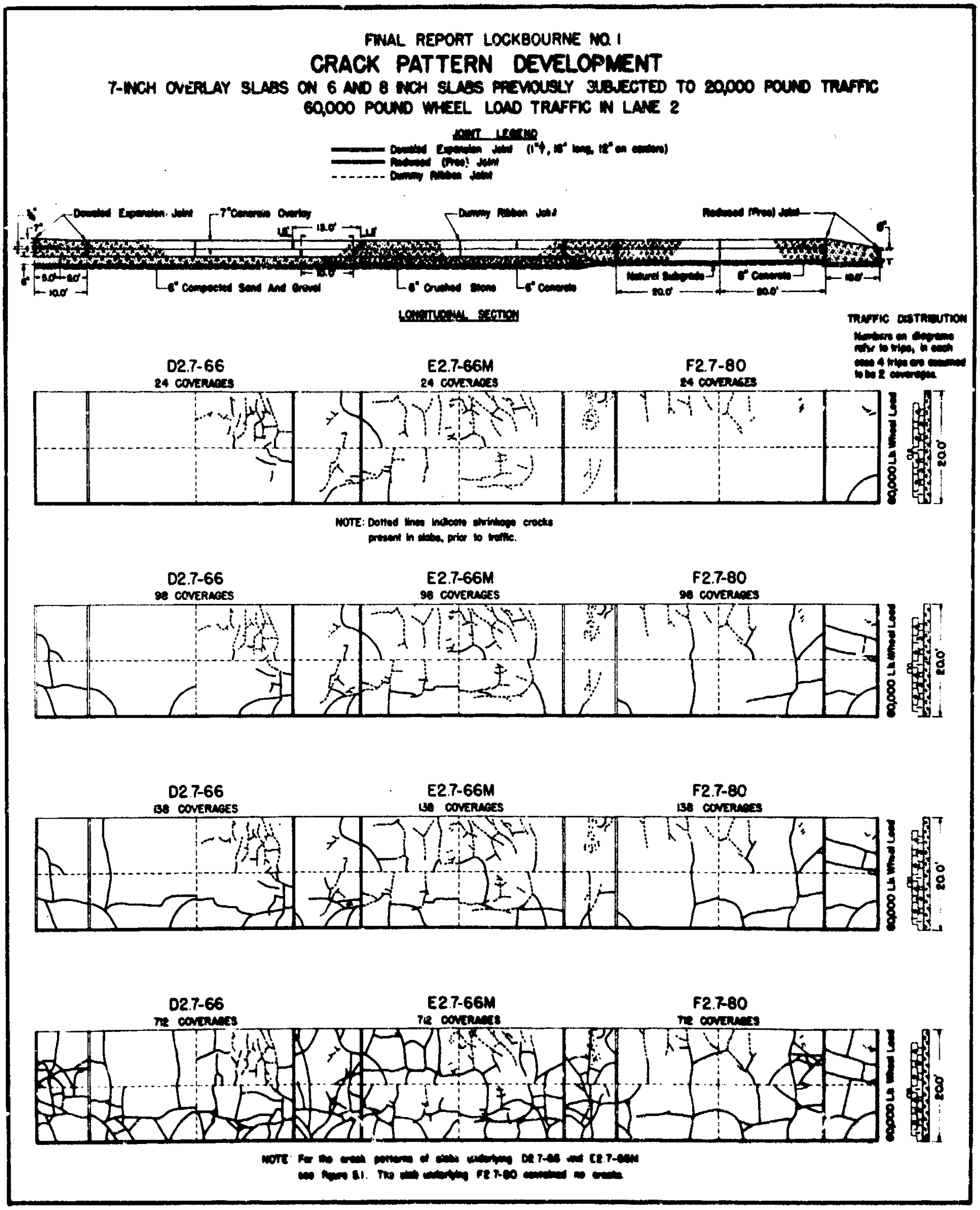

FINAL REPORT LOCKBOURNE NO. I
CRACK PATTERN DEVELOPMENT
7-INCH OVERLAY SLABS ON 6 AND 8 INCH SLABS PREVIOUSLY SUBJECTED TO 20,000 POUND TRAFFIC
60,000 POUND WHEEL LOAD TRAFFIC IN LANE 2
7" Concrete Overlay
6" Compacted Sand And Gravel
6" Crushed Stone
6" Concrete
Natural Subgrade
8" Concrete
LONGITUDINAL SECTION
TRAFFIC DISTRIBUTION
D2.7-66
24 COVERAGES
E2.7-66M
24 COVERAGES
F2.7-80
24 COVERAGES
NOTE: Dotted lines indicate shrinkage cracks present in slabs, prior to traffic.
D2.7-66
98 COVERAGES
E2.7-66M
98 COVERAGES
F2.7-80
98 COVERAGES
D2.7-66
138 COVERAGES
E2.7-66M
138 COVERAGES
F2.7-80
138 COVERAGES
D2.7-66
712 COVERAGES
E2.7-66M
712 COVERAGES
F2.7-80
712 COVERAGES
60,000 Lb Wheel Load
20.0'

FIGURE 5.10

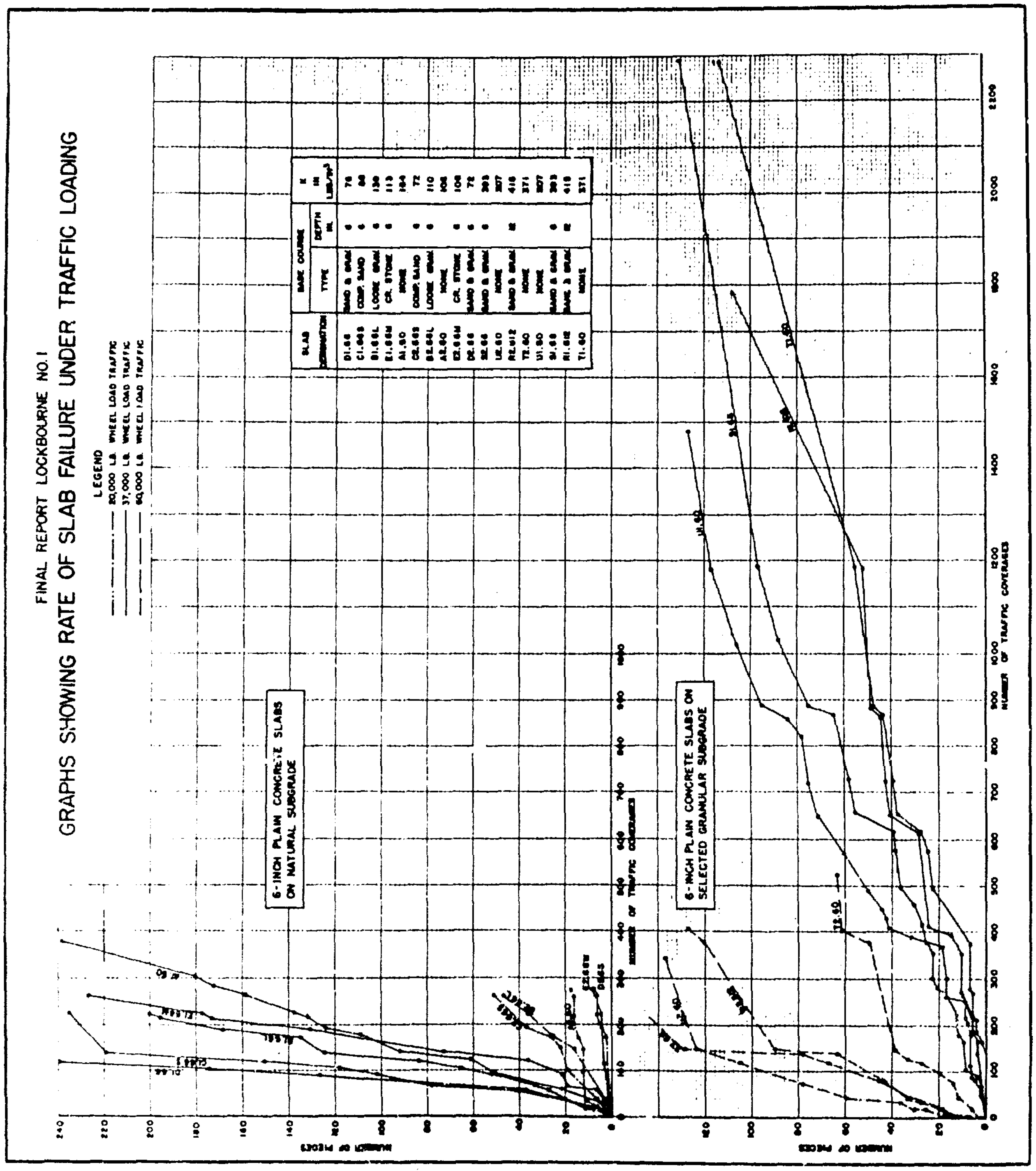
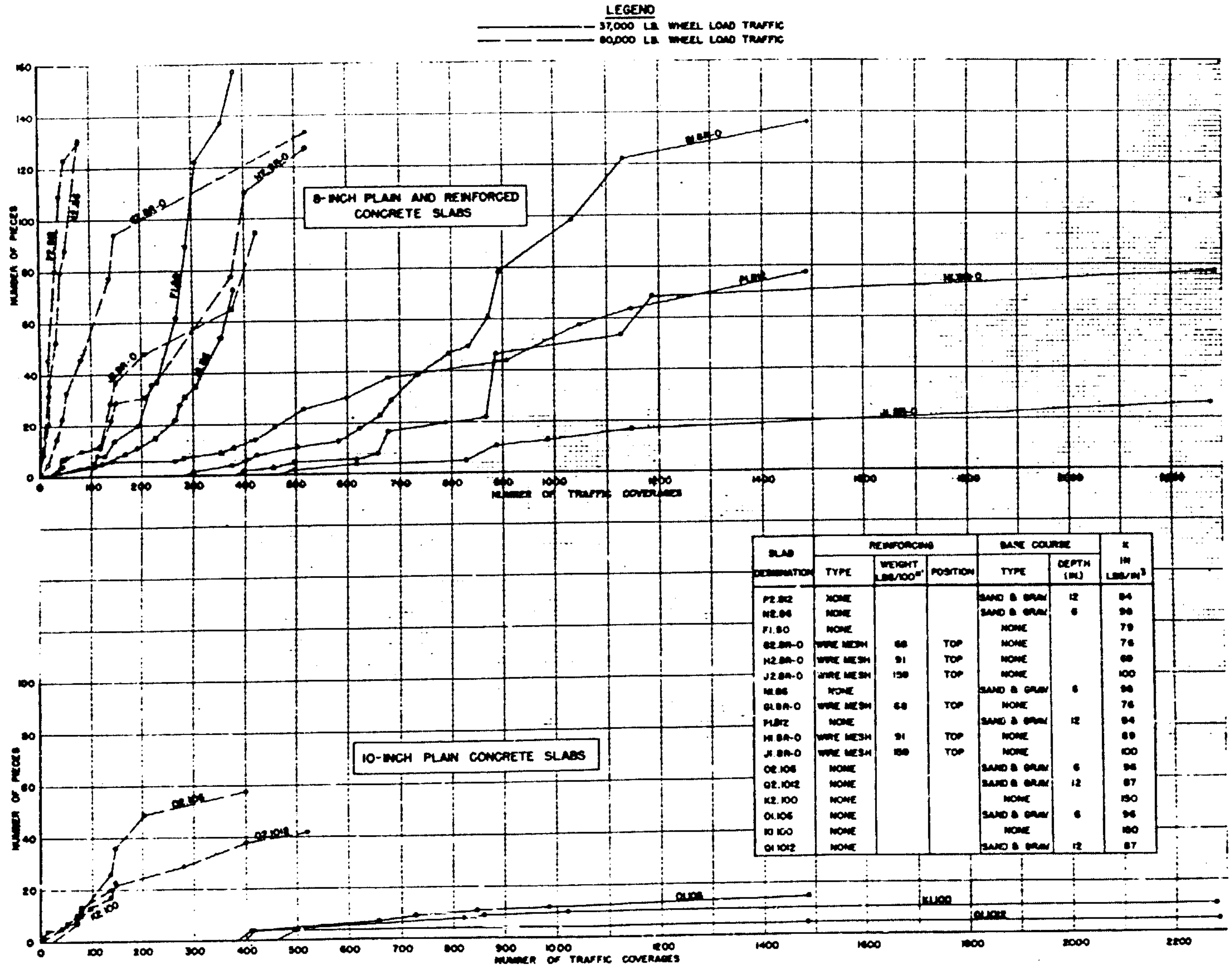

FINAL REPORT LOCKBOURNE NO. 1
GRAPHS SHOWING RATE OF SLAB FAILURE UNDER TRAFFIC LOADING
LEGEND
20,000 LB. WHEEL LOAD TRAFFIC
37,000 LB. WHEEL LOAD TRAFFIC
60,000 LB. WHEEL LOAD TRAFFIC
6-INCH PLAIN CONCRETE SLABS ON NATURAL SUBGRADE
6-INCH PLAIN CONCRETE SLABS ON SELECTED GRANULAR SUBGRADE
NUMBER OF TRAFFIC COVERAGES
NUMBER OF PIECES

FIGURE 6.0

FINAL REPORT LOCKBOURNE NO. 1

GRAPHS SHOWING RATE OF SLAB FAILURE UNDER TRAFFIC LOADING



SLAB DESIGNATION	REINFORCING TYPE	REINFORCING WEIGHT LBS/100 FT²	REINFORCING POSITION	BASE COURSE TYPE	BASE COURSE DEPTH (IN.)	K IN LBS/IN³
F2.812	NONE			SAND & GRAV	12	94
H2.86	NONE			SAND & GRAV	6	96
F1.80	NONE			NONE		79
G2.8R-0	WIRE MESH	68	TOP	NONE		76
H2.8R-0	WIRE MESH	91	TOP	NONE		69
J2.8R-0	WIRE MESH	150	TOP	NONE		100
H1.86	NONE			SAND & GRAV	6	96
G1.8R-0	WIRE MESH	68	TOP	NONE		76
F1.812	NONE			SAND & GRAV	12	94
H1.8R-0	WIRE MESH	91	TOP	NONE		69
J1.8R-0	WIRE MESH	150	TOP	NONE		100
O2.106	NONE			SAND & GRAV	6	96
Q2.1012	NONE			SAND & GRAV	12	87
K2.100	NONE			NONE		150
O1.106	NONE			SAND & GRAV	6	96
K1.100	NONE			NONE		160
Q1.1012	NONE			SAND & GRAV	12	87

FIGURE 6.1

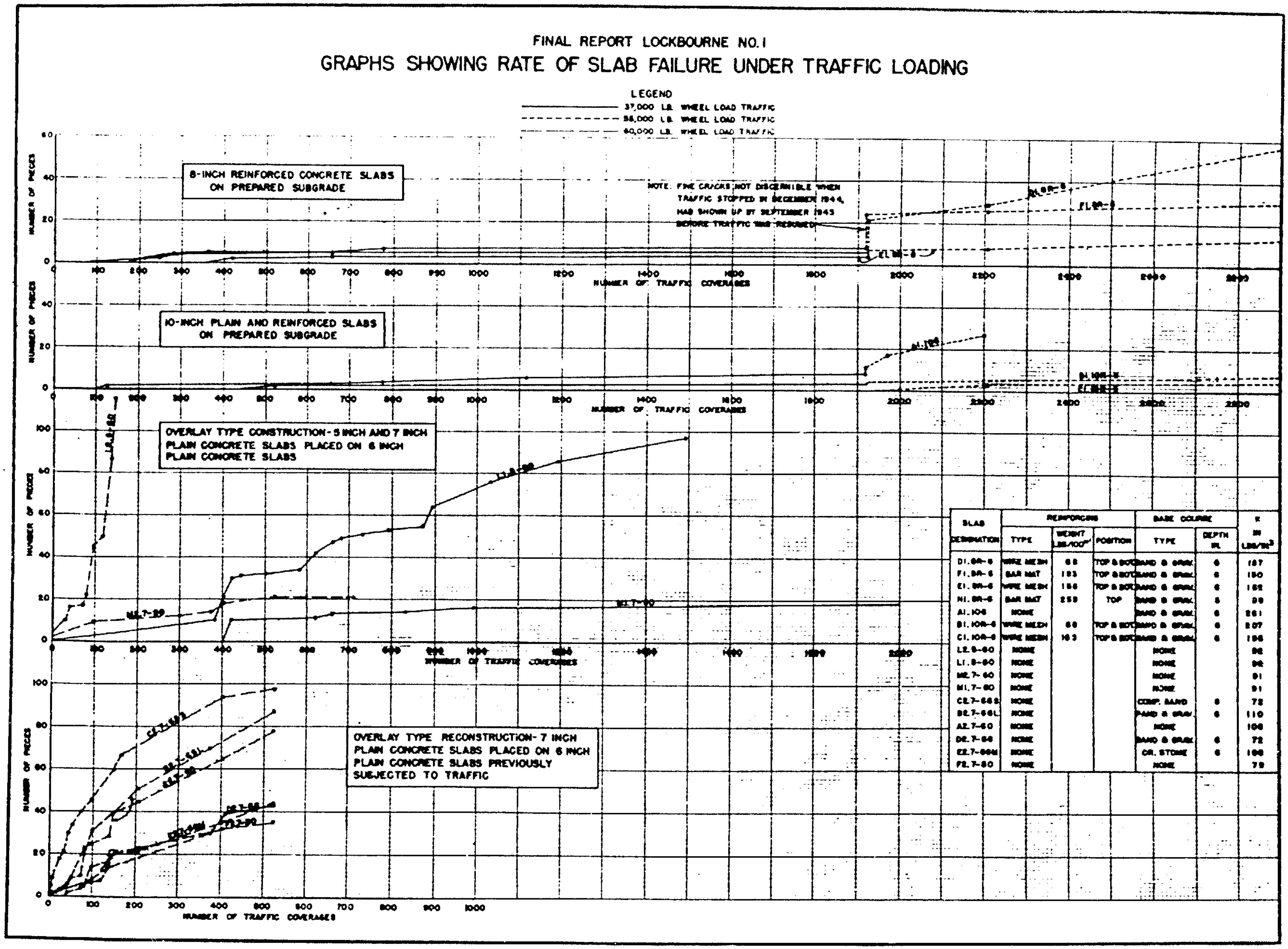
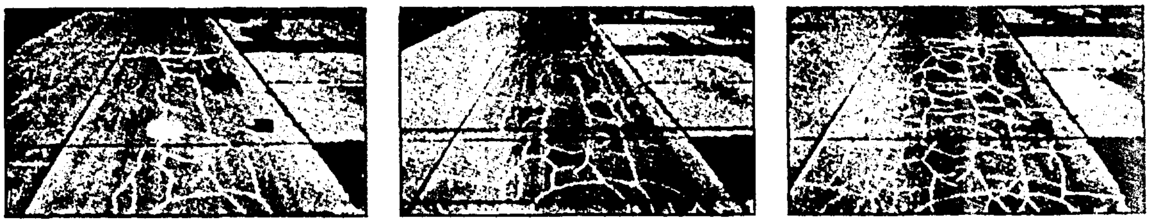

FINAL REPORT LOCKBOURNE NO. I
GRAPHS SHOWING RATE OF SLAB FAILURE UNDER TRAFFIC LOADING
LEGEND
37,000 LB. WHEEL LOAD TRAFFIC
56,000 LB. WHEEL LOAD TRAFFIC
60,000 LB. WHEEL LOAD TRAFFIC
8-INCH REINFORCED CONCRETE SLABS ON PREPARED SUBGRADE
NOTE: FINE CRACKS NOT DISCERNIBLE WHEN TRAFFIC STOPPED IN DECEMBER 1944, HAD SHOWN UP BY SEPTEMBER 1945 BEFORE TRAFFIC WAS RESUMED.
10-INCH PLAIN AND REINFORCED SLABS ON PREPARED SUBGRADE
OVERLAY TYPE CONSTRUCTION - 5 INCH AND 7 INCH PLAIN CONCRETE SLABS PLACED ON 6 INCH PLAIN CONCRETE SLABS
OVERLAY TYPE RECONSTRUCTION - 7 INCH PLAIN CONCRETE SLABS PLACED ON 6 INCH PLAIN CONCRETE SLABS PREVIOUSLY SUBJECTED TO TRAFFIC
NUMBER OF PIECES
NUMBER OF TRAFFIC COVERAGES
SLAB DESIGNATION
REINFORCING
BASE COURSE

FIGURE 6.2

FINAL REPORT LOCKBOURNE NO. I

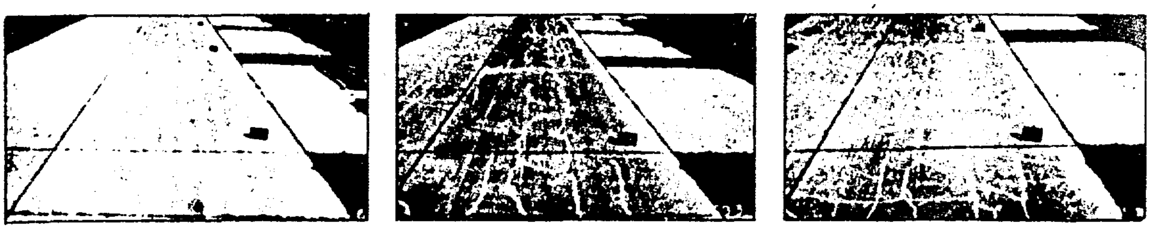
CRACK DEVELOPMENT IN 6 INCH SLABS ON DEEP GRANULAR SUBGRADES
60,000 POUND WHEEL LOAD TRAFFIC

24 COVERAGES | 80 COVERAGES | 158 COVERAGES

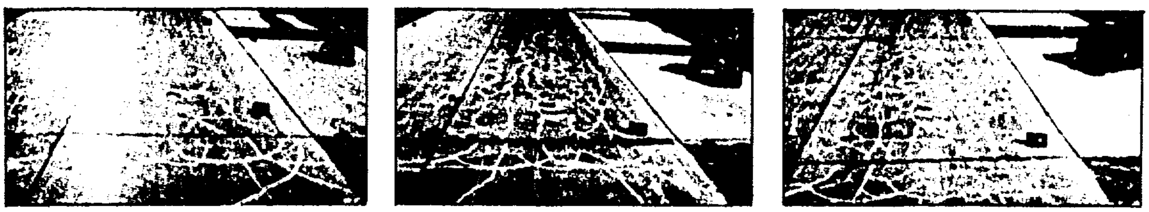


6" CONCRETE ON 12" COMPACTED SAND AND GRAVEL ON 60" SOIL STABILIZED SAND AND GRAVEL
SLAB R2.612

6" CONCRETE ON 6" COMPACTED SAND AND GRAVEL ON 66" SOIL STABILIZED SAND AND GRAVEL
SLAB S2.66



6" CONCRETE ON 72" COMPACTED SAND AND GRAVEL
SLAB T2.60



6" CONCRETE ON 72" COMPACTED SAND
SLAB U2.60

FIGURE 7.0

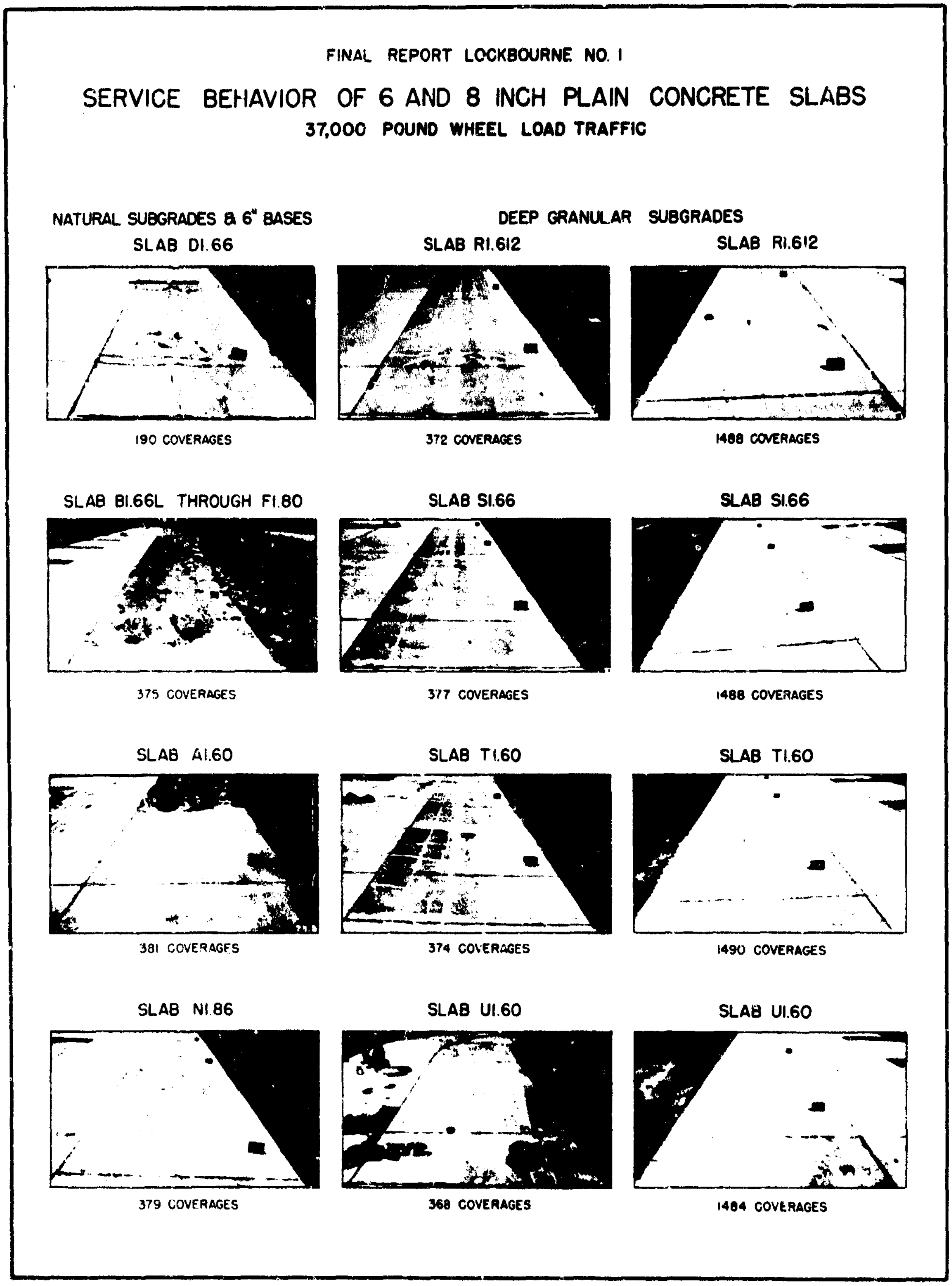

FINAL REPORT LOCKBOURNE NO. I
SERVICE BEHAVIOR OF 6 AND 8 INCH PLAIN CONCRETE SLABS
37,000 POUND WHEEL LOAD TRAFFIC
NATURAL SUBGRADES & 6" BASES
DEEP GRANULAR SUBGRADES
SLAB DI.66
SLAB RI.612
SLAB RI.612
190 COVERAGES
372 COVERAGES
1488 COVERAGES
SLAB BI.66L THROUGH FI.80
SLAB SI.66
SLAB SI.66
375 COVERAGES
377 COVERAGES
1488 COVERAGES
SLAB AI.60
SLAB TI.60
SLAB TI.60
381 COVERAGES
374 COVERAGES
1490 COVERAGES
SLAB NI.86
SLAB UI.60
SLAB UI.60
379 COVERAGES
368 COVERAGES
1484 COVERAGES

FIGURE 7.

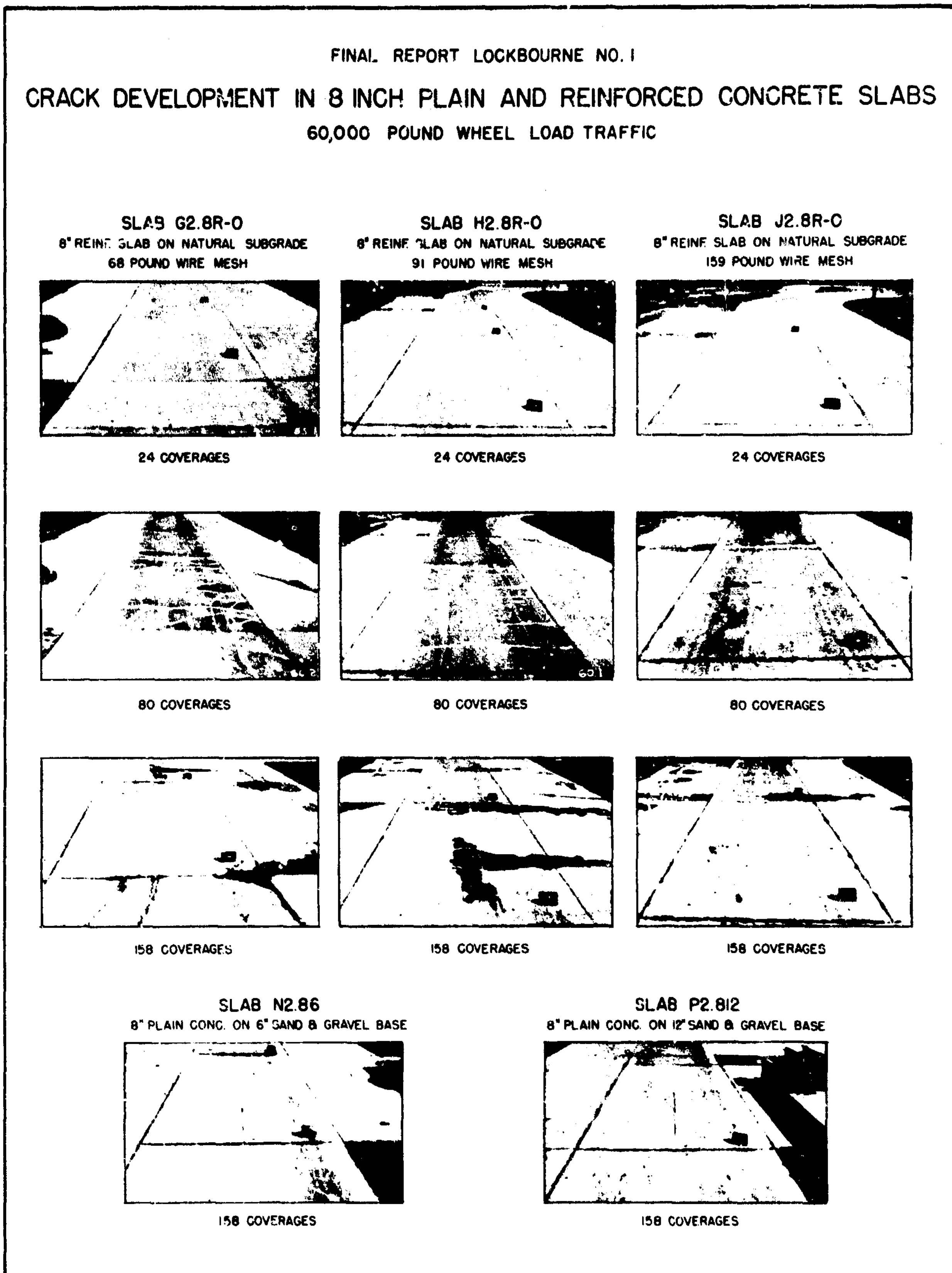

FINAL REPORT LOCKBOURNE NO. I
CRACK DEVELOPMENT IN 8 INCH PLAIN AND REINFORCED CONCRETE SLABS
60,000 POUND WHEEL LOAD TRAFFIC
SLAB G2.8R-0
8" REINF. SLAB ON NATURAL SUBGRADE
68 POUND WIRE MESH
SLAB H2.8R-0
8" REINF. SLAB ON NATURAL SUBGRADE
91 POUND WIRE MESH
SLAB J2.8R-0
8" REINF. SLAB ON NATURAL SUBGRADE
159 POUND WIRE MESH
24 COVERAGES
24 COVERAGES
24 COVERAGES
80 COVERAGES
80 COVERAGES
80 COVERAGES
158 COVERAGES
158 COVERAGES
158 COVERAGES
SLAB N2.86
8" PLAIN CONC. ON 6" SAND & GRAVEL BASE
158 COVERAGES
SLAB P2.812
8" PLAIN CONC. ON 12" SAND & GRAVEL BASE
158 COVERAGES

FIGURE 7.2

FINAL REPORT LOCKBOURNE NO. 1

SERVICE BEHAVIOR OF 8 AND 10 INCH PLAIN AND REINFORCED CONCRETE SLABS ON PREPARED SUBGRADE

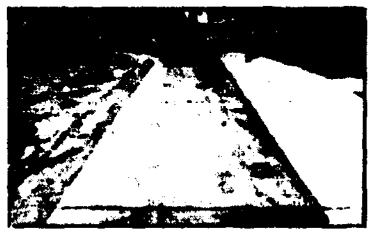
37,000 AND 55,000 POUND WHEEL LOAD TRAFFIC

SLAB A1.10G
10" PLAIN CONCRETE

758 COVERAGES - 37,000 LB. W.L. TRAFFIC

1918 COVERAGES - 37,000 LB. W.L. TRAFFIC
284 COVERAGES - 55,000 LB. W.L. TRAFFIC

SLAB B1.10R-6
10" REINFORCED CONCRETE
68 LB. WIRE MESH, TOP & BOTTOM

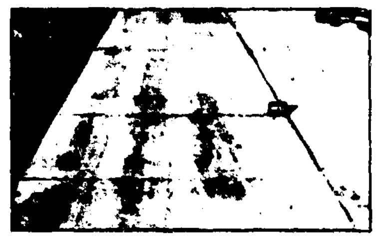


763 COVERAGES - 37,000 LB. W.L. TRAFFIC

1920 COVERAGES - 37,000 LB. W.L. TRAFFIC
1084 COVERAGES - 55,000 LB. W.L. TRAFFIC

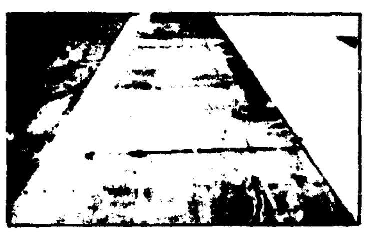
SLAB C1.10R-6
10" REINFORCED CONCRETE
156 LB. WIRE MESH, TOP & BOTTOM

763 COVERAGES - 37,000 LB. W.L. TRAFFIC



1923 COVERAGES - 37,000 LB. W.L. TRAFFIC
1084 COVERAGES - 55,000 LB. W.L. TRAFFIC

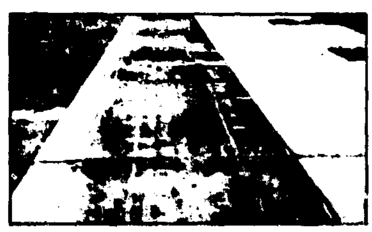
SLAB D1.8R-6
8" REINFORCED CONCRETE
68 LB. WIRE MESH, TOP & BOTTOM



760 COVERAGES - 37,000 LB. W.L. TRAFFIC

1923 COVERAGES - 37,000 LB. W.L. TRAFFIC
1084 COVERAGES - 55,000 LB. W.L. TRAFFIC

SLAB E1.8R-6
8" REINFORCED CONCRETE
156 LB. WIRE MESH, TOP & BOTTOM

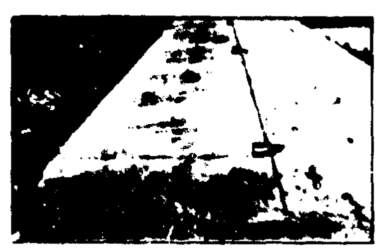


761 COVERAGES - 37,0[illegible] W.L. TRAFFIC

1923 COVERAGES - 37,000 LB. W.L. TRAFFIC
1084 COVERAGES - 55,000 LB. W.L. TRAFFIC

SLAB F1.8R-6
8" REINFORCED CONCRETE
1/2" DEF BARS 8" O.C. BOTH WAYS TOP & BOT.

758 COVERAGES - 37,000 LB. W.L. TRAFFIC



1918 COVERAGES - 37,000 LB. W.L. TRAFFIC
1084 COVERAGES - 55,000 LB. W.L. TRAFFIC

FIGURE 7.3